Technical Report CERC-93-19
December 1993

US Army Corps of Engineers
Waterways Experiment Station

Engineering Design Guidance for Detached Breakwaters as Shoreline Stabilization Structures

by Monica A. Chasten, Julie D. Rosati, John W. McCormick
Coastal Engineering Research Center

Robert E. Randall
Texas A&M University

Prepared for Headquarters, U.S. Army Corps of Engineers

Technical Report CERC-93-19
December 1993

Engineering Design Guidance for Detached Breakwaters as Shoreline Stabilization Structures

by Monica A. Chasten, Julie D. Rosati, John W. McCormick
Coastal Engineering Research Center

U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

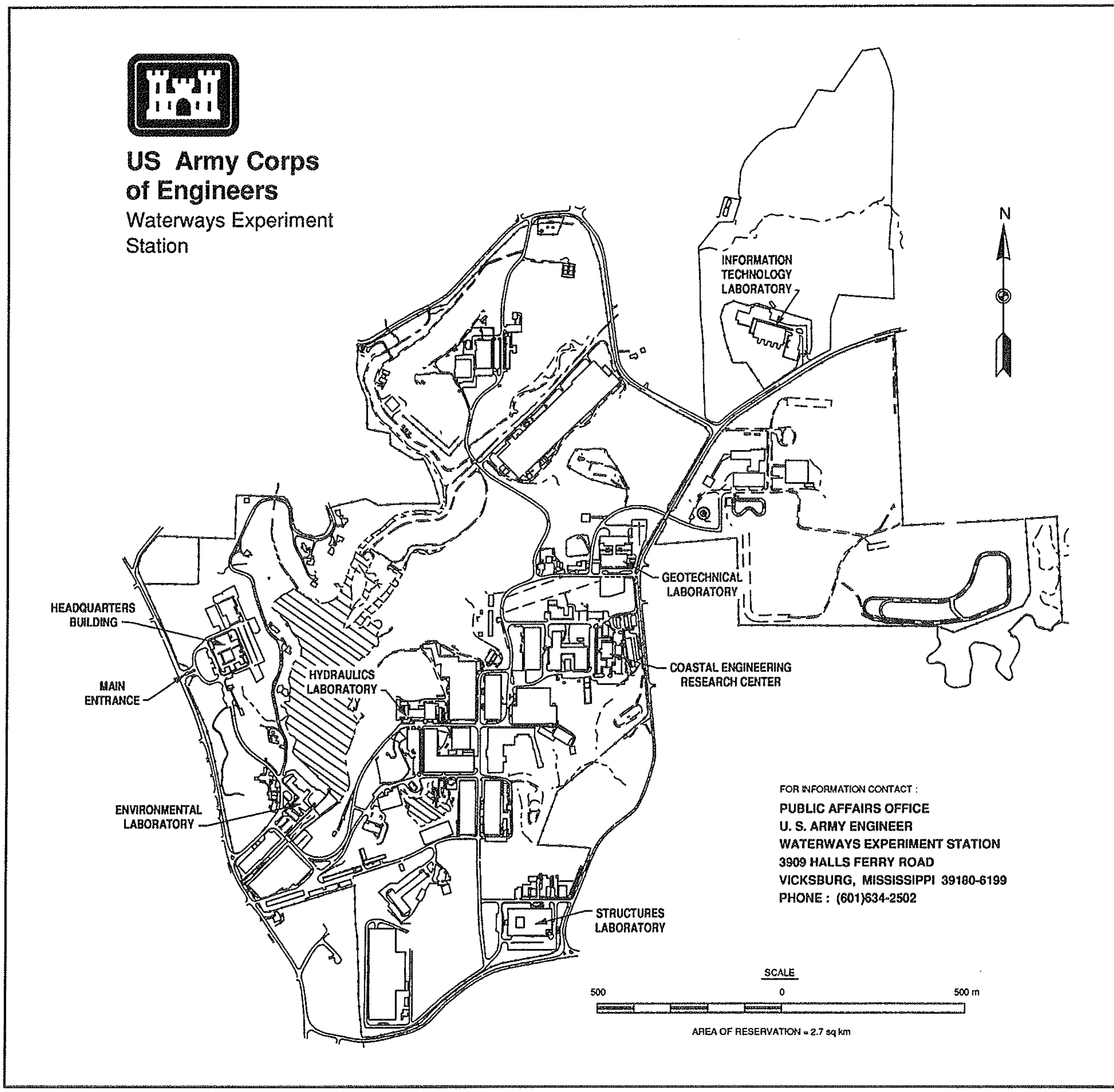
Dr. Robert E. Randall

Texas A&M University
Ocean Engineering Program
Civil Engineering Department
College Station, TX 77843

Final report

Prepared for U.S. Army Corps of Engineers
Washington, DC 20314-1000

Under Work Unit 32748



Waterways Experiment Station Cataloging-in-Publication Data

Engineering design guidance for detached breakwaters as shoreline stabilization structures / by Monica A. Chasten ... [et al.], Coastal Engineering Research Center ; prepared for U.S. Army Corps of Engineers.
167 p. : ill. ; 28 cm. — (Technical report ; CERC-93-19)
Includes bibliographical references.
1. Breakwaters — Design and construction. 2. Shore protection. 3. Coastal engineering. I. Chasten, Monica A. II. United States. Army. Corps of Engineers. III. Coastal Engineering Research Center (U.S.) IV. U.S. Army Engineer Waterways Experiment Station. V. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; CERC-93-19.
TA7 W34 no.CERC-93-19

Contents

List of Figures

List of Tables

Preface

This report was authorized as a part of the Civil Works Research and Development Program by Headquarters, U.S. Army Corps of Engineers (HQUSACE). The work was conducted under Work Unit 32748, "Detached Breakwaters for Shoreline Stabilization," under the Coastal Structure Evaluation and Design Program at the Coastal Engineering Research Center (CERC), U.S. Army Engineer Waterways Experiment Station (WES). Messrs. J. H. Lockhart and J. G. Housley were HQUSACE Technical Monitors.

This report was prepared by Ms. Monica A. Chasten, Coastal Structures and Evaluation Branch (CSEB), CERC, Ms. Julie D. Rosati, Coastal Processes Branch (CPB), CERC, Mr. John W. McCormick, CSEB, CERC, and Dr. Robert E. Randall, Texas A&M University. Mr. Edward T. Fulford of Andrews Miller and Associates, Inc. prepared Appendix A. This report was technically reviewed by Dr. Yen-hsi Chu, Chief, Engineering Applications Unit, CSEB, CERC, Mr. Mark Gravens, CPB, CERC, Dr. Nicholas Kraus, formerly of CERC, and Mr. John P. Ahrens, National Sea Grant College Program, National Oceanic and Atmospheric Administration. Ms. Kelly Lanier and Ms. Janie Daughtry, CSEB, CERC, assisted with final report preparation. The study was conducted under the general administrative supervision of Dr. Yen-hsi Chu, Ms. Joan Pope, Chief, CSEB, CERC, and Mr. Thomas W. Richardson, Chief, Engineering Development Division, CERC. Director of CERC during the investigation was Dr. James R. Houston, and Assistant Director was Mr. Charles C. Calhoun, Jr.

Director of WES during publication of this report was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
cubic yards	0.7645549	cubic meters
degrees (angle)	0.01745329	radians
pounds (mass)	0.4535924	kilograms
knots	0.514444	meters per second
nautical miles	1.852	kilometers
cubic feet	0.02831685	cubic meters
miles	1.609347	kilometers

1 Introduction

With increased use and development of the coastal zone, beach erosion in some areas may become serious enough to warrant the use of protective coastal structures. Based on prototype experience, detached breakwaters can be a viable method of shoreline stabilization and protection in the United States. Breakwaters can be designed to retard erosion of an existing beach, promote natural sedimentation to form a new beach, increase the longevity of a beach fill, and maintain a wide beach for storm damage reduction and recreation. The combination of low-crested breakwaters and planted marsh grasses is increasingly being used to establish wetlands and control erosion along estuarine shorelines.

General Description

Detached breakwaters are generally shore-parallel structures that reduce the amount of wave energy reaching the protected area by dissipating, reflecting, or diffracting incoming waves. The structures dissipate wave energy similar to a natural offshore bar, reef, or nearshore island. The reduction of wave action promotes sediment deposition shoreward of the structure. Littoral material is deposited and sediment retained in the sheltered area behind the breakwater. The sediment will typically appear as a bulge in the beach planform termed a salient, or a tombolo if the resulting shoreline extends out to the structure (Figure 1).

Breakwaters can be constructed as a single structure or in series. A single structure is used to protect a localized project area, whereas a multiple segment system is designed to protect an extended length of shoreline. A segmented system consists of two or more structures separated by gaps with specified design widths.

Unlike shore-perpendicular structures, such as groins, which may impound sediment, properly designed breakwaters can allow continued movement of longshore transport through the project area, thus reducing adverse impacts on downdrift beaches. Effects on adjacent shorelines are further minimized when beach fill is included in the project. Some disadvantages associated with

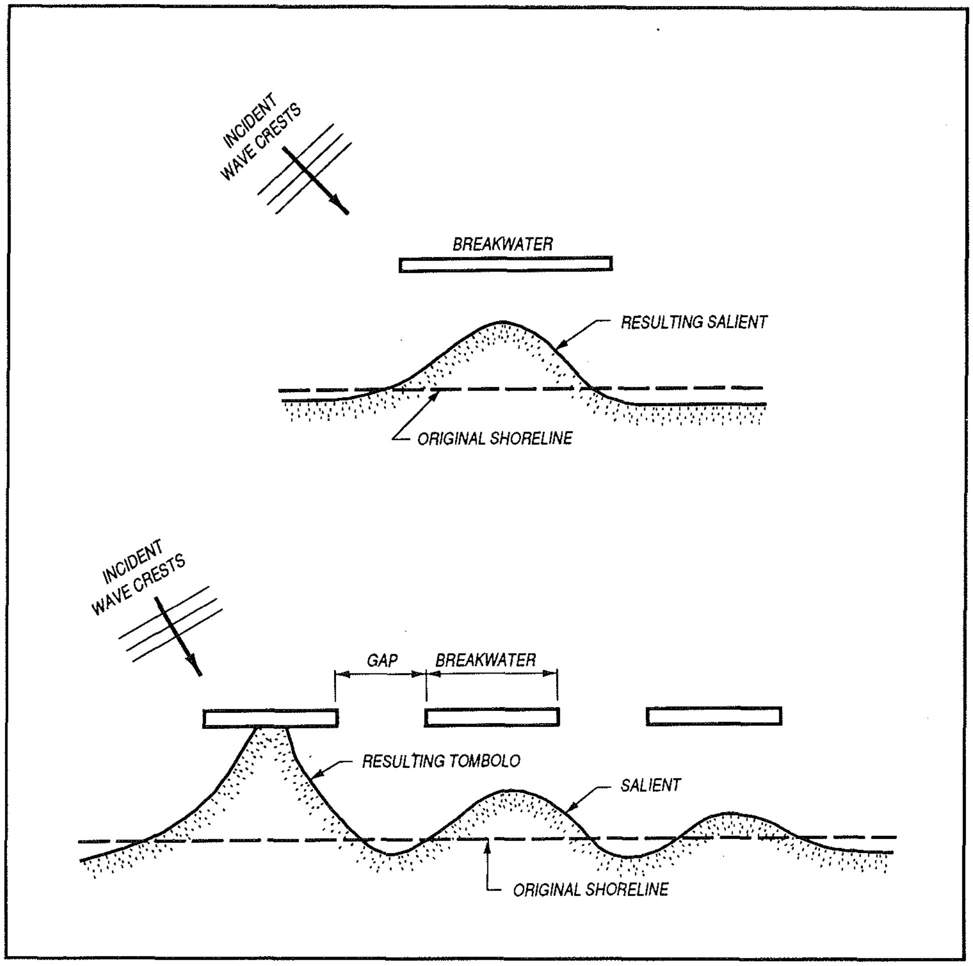


Figure 1. Types of shoreline changes associated with single and multiple breakwaters and definition of terminology (modified from EM 1110-2-1617)

detached breakwaters include limited design guidance, high construction costs, and a limited ability to predict and compensate for structure-related phenomena such as adjacent beach erosion, rip currents, scour at the structure's base, structure transmissibility, and effects of settlement on project performance.

Breakwater Types

There are numerous variations of the breakwater concept. Detached breakwaters are constructed at a significant distance offshore and are not connected to shore by any type of sand-retaining structure. Reef breakwaters are a type of detached breakwater designed with a low crest elevation and homogeneous stone size, as opposed to the traditional multilayer cross section. Low-crested breakwaters can be more suitable for shoreline stabilization projects due to increased tolerance of wave transmission and reduced quantities of material necessary for construction. Other types of breakwaters include headland breakwaters or artificial headlands, which are constructed at or very near to the original shoreline. A headland breakwater is designed to promote beach growth out to the structure, forming a tombolo or periodic tombolo, and tends to function as a transmissible groin (Engineer Manual (EM) 1110-2-1617, Pope 1989). Another type of shore-parallel offshore structure is called a submerged sill or perched beach. A submerged or semi-submerged sill reduces the rate of offshore sand movement from a stretch of beach by acting as a barrier to shore-normal transport. The effect of submerged sills on waves is relatively small due to their low crest elevation (EM 1110-2-1617). Other types of shore-parallel structures include numerous patented commercial systems, which have had varying degrees of efficiencies and success rates. This technical report will focus on detached breakwater design guidance for shoreline stabilization purposes and provide a general discussion of recently constructed headland and low-crested breakwater projects. Additional information and references on other breakwater classifications can be found in Lesnik (1979), Bishop (1982), Fulford (1985), Pope (1989), and EM 1110-2-1617.

Prototype Experience

Prototype experience with detached breakwaters as shore protection structures in the United States has been limited. Twenty-one detached breakwater projects, 225 segments, exist along the continental U.S. and Hawaiian coasts, including 76 segments recently constructed near Peveto and Holly Beach, Louisiana, and another 55 segments completed in 1992 at Presque Isle, Pennsylvania (Figure 2). Comparatively, at least 4,000 detached breakwater segments exist along Japan's 9,400-km coastline (Rosati and Truitt 1990). Breakwaters have been used extensively for shore protection in Japan and Israel (Toyoshima 1976, 1982; Goldsmith 1990), in low to moderate wave energy environments with sediment ranging from fine sand to pebbles. Other countries with significant experience in breakwater design and use include Spain, Denmark, and Singapore (Rosati 1990). Figures 3 to 5 show various examples of international breakwater projects.

United States experience with segmented detached breakwater projects has been generally limited to littoral sediment-poor shorelines characterized by a local fetch-dominated wave climate (Pope and Dean 1986). Most projects are located on the Great Lakes, Chesapeake Bay, or Gulf of Mexico shorelines. These projects are typically subjected to short-period, steep waves, which tend to approach the shoreline with limited refraction, and generally break at steep angles to the shoreline. The projects also tend to be in areas that are prone to storm surges and erratic water level fluctuations, particularly in the Great Lakes regions.

In recent years, low-crested breakwaters of varied types have been used in conjunction with marsh grass plantings in an attempt to create and/or stabilize

Figure 2. Segmented detached breakwaters at Presque Isle, Pennsylvania, on Lake Erie, fall 1992

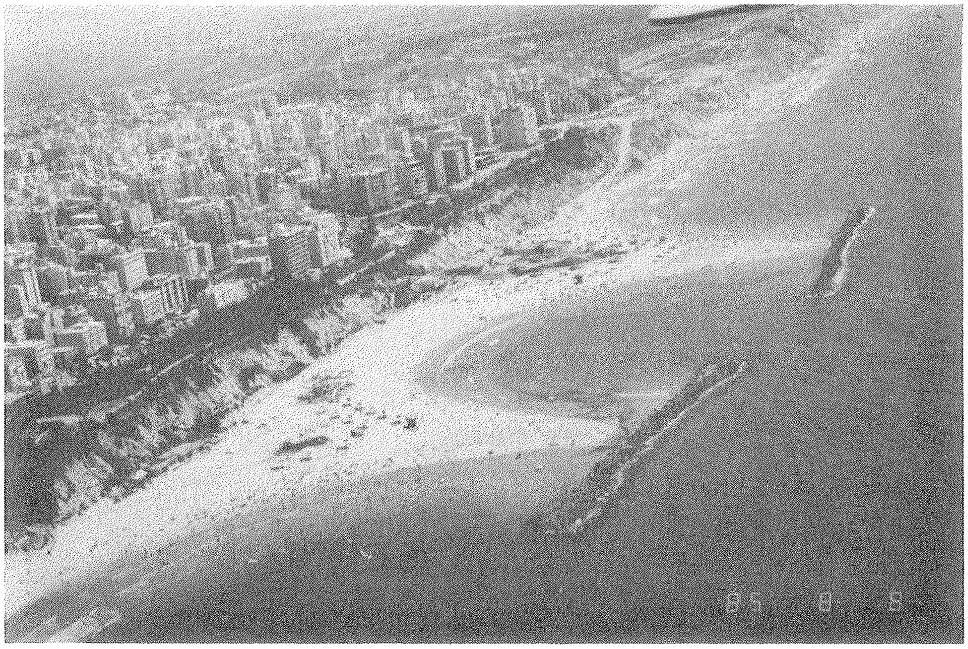


Figure 3. Detached breakwaters in Netanya, Israel, August 1985 (from Goldsmith (1990))

Figure 4. Segmented detached breakwaters in Japan

Figure 5. Detached breakwater project in Spain

wetland areas (Landin, Webb, and Knutson 1989; Rogers 1989; Knutson, Allen, and Webb 1990; EM 1110-2-5026). Recent wetland/breakwater projects include Eastern Neck, Maryland (Figure 6) constructed by the U.S. Fish and Wildlife Service with dredge material provided by the U.S. Army Engineer District (USAED), Baltimore; and Aransas, Texas, presently under construction and developed by the USAED, Galveston, and the U.S. Army Engineer Waterways Experiment Station (WES) Coastal Engineering Research Center (CERC).

Detailed summaries of the design and performance of single and segmented detached breakwater projects in the United States have been provided in a number of references (Dally and Pope 1986, Pope and Dean 1986, Kraft and Herbich 1989). Table 1 provides a summary of a number of detached breakwater projects. Most recently constructed breakwater projects have been located on the Great Lakes or Chesapeake Bay (Figure 7) (Hardaway and Gunn 1991a and 1991b, Mohr and Ippolito 1991, Bender 1992, Coleman 1992, Fulford and Usab 1992). A number of private breakwater projects have been constructed, but are not shown in Table 1.

Existing Design Guidance

Internationally and throughout the United States various schools of thought have emerged on the design and construction of breakwaters (Pope 1989). Japanese and U.S. projects tend to vary in style within each country, but often use the segmented detached breakwater concept. In Denmark, Singapore,

Table 1
Summary of U.S. Breakwater Projects

Coast	Project	Location	Date of Construction	Number of Segments	Project Length	Segment Length	Gap Length	Distance Offshore Preproject	Water Depth	Fill Placed	Beach* Response	Constructed by	Maintained by
Atlantic	Winthrop Beach (low tide)	Massachusetts	1935	5	625m	91m	30m	Unknown	3.0m (mlw)	No	1	State of Mass.	
Atlantic	Winthrop Beach (high tide)	Massachusetts	1935	1		100	30	305	3.0 (mhw)	No	3	State of Mass.	
Atlantic (Potomac River)	Colonial Beach (Central Beach)	Virginia	1982	4	427	61	46	64	1.2	Yes	2	USACE	
Atlantic (Potomac River)	Colonial Beach (Castlewood Park)	Virginia	1982	3	335	61,93	26,40	46	1.2	Yes	1	USACE	
Chesapeake Bay	Elm's Bèach (wetland)	Maryland	1985	3	335	47	53	44	0.6-0.9	Yes	1	State of Maryland	
Chesapeake Bay	Elk Neck State Park (wetland)	Maryland	1989	4	107	15	15		0.6-0.9	No	2-4	USACE	USACE
Chesapeake Bay	Terrapin Beach (wetland)	Maryland	1989	4		23	15,31,23	38.1	0.6-0.9	Yes	5	USACE	USACE
Chesapeake Bay	Eastern Neck (wetland)	Maryland	1992-1993	26	1676	31	23		0.3-0.6	Yes		US Fish and Wildlife Service, USACE	US Fish and Wildlife Service
Chesapeake Bay	Bay Ridge	Maryland	1990-1991	11	686	31	31	42.7		Yes	4	Private	Private
Gulf of Mexico	Redington Shores	Florida	1985-1986	1	100	100	0	104		Yes	1	USACE	USACE
Gulf of Mexico	Holly Beach	Louisiana	1985	6	555	46,51,50	93,89	78,61	2.5	No	4	State of Louisiana	State of Louisiana
Gulf of Mexico	Holly Beach	Louisiana	1991-1993	76		46,53	91,84	122,183	1.4,1.6	Yes	3	State of Louisiana	State of Louisiana
Gulf of Mexico	Grand Isle	Louisiana		4	84	70	21	107	2	No	3	City of Grand Isle	City of Grand Isle
Lake Erie	Lakeview Park	Ohio	1977	3	403	76	49	152	3.7	Yes	4	USACE	City of Lorain
Lake Erie	Presque Isle	Pennsylvania	1978	3	440	38	61,91	60	0.9-1.2	Yes	2	USACE	USACE
Lake Erie	Presque Isle	Pennsylvania	1989-1992	55	8300	46	107	76-107	1.5-2.4 (lwd)	Yes	3-4	USACE	USACE
Lake Erie	Lakeshore Park	Ohio	1982	3	244	38	61	120	2.1	Yes	5	USACE	City of Ashtabula
Lake Erie	East Harbor	Ohio	1983	4	550	46	90,105,120	170	2.3	No	5	State of Ohio	State of Ohio
Lake Erie	Maumee Bay (headland)	Ohio	1990	5	823	61	76		1.3	Yes	1	USACE	State of Ohio
Lake Erie	Sims Park (headland)	Ohio	1992	3	975	38	49		2.5	Yes	1	USACE	City of Euclid
Pacific	Venice	California	1905	1	180	180	0	370		No	5	Private	
Pacific	Haleiwa Beach	Hawaii	1965	1	49	49	0	90	2.1 (msl)	Yes	3	USACE/State of HI	USACE
Pacific	Sand Island	Hawaii	1991	3	110	21	23					USACE	USACE

*Beach response is coded as follows: 1-permanent tombolos, 2-periodic tombolos, 3-well developed salients, 4-subdued salients, 5-no sinuosity

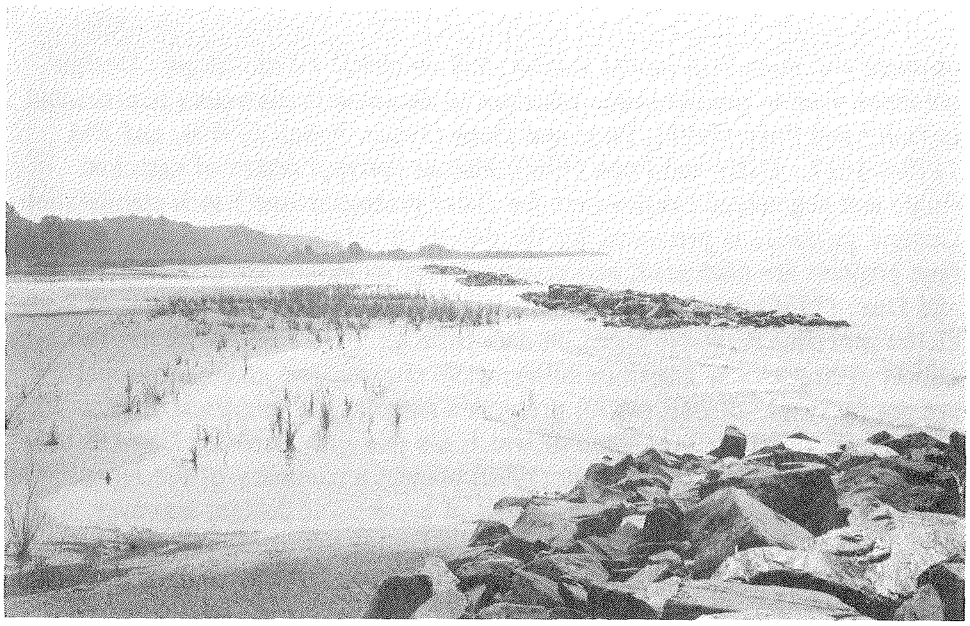


Figure 6. Breakwaters constructed for wetland development at Eastern Neck, Maryland

Figure 7. Detached breakwaters constructed on Chesapeake Bay at Bay Ridge, Maryland

Spain, and some projects along the U.S. Great Lakes and eastern estuarine shorelines, the trend is towards artificial headland systems. Along the Chesapeake Bay, the use of low-crested breakwaters has become popular since they can be more cost-effective and easier to contruct than traditional multilayered breakwaters.

Previous U.S. Army Corps of Engineers (USACE) breakwater projects have been designed based on the results of existing prototype projects, physical and numerical model studies, and empirical relationships. Design guidance used to predict beach response to detached breakwaters is presented in Dally and Pope (1986), Pope and Dean (1986), Rosati (1990), and EM 1110-2-1617. Dally and Pope (1986) discuss the application of detached single and segmented breakwaters for shore protection and beach stabilization. General guidance is presented for the design of detached breakwaters, prototype projects are discussed, and several design examples are provided. Pope and Dean (1986) present a preliminary design relationship with zones of predicted shoreline response based on data from ten field sites; however, the effects of breakwater transmissibility, wave climate, and sediment properties are not included. Rosati (1990) presents a summary of empirical relationships available in the literature, some of which are presently used for USACE breakwater design. Rosati and Truitt (1990) present a summary of the Japanese Ministry of Construction (JMC) method of breakwater design; however, this method has not been frequently used in the United States. Guidance on Japanese design methods is also provided in Toyoshima (1974). Engineer Manual 1110-2-1617, *Coastal Groins and Nearshore Breakwaters*, contains the most recent USACE design guidance for breakwaters. This manual provides guidelines and design concepts for beach stabilization structures, including detached breakwaters, and provides appropriate references for available design procedures. Although numerous references exist for functional design of U.S. detached breakwater projects, the predictive ability for much of this guidance is limited. Knowledge of coastal processes at a project site, experience from other prototype projects, and a significant amount of engineering judgement must be incorporated in the functional design of a breakwater project.

Design guidance on the use of low-crested rubble-mound breakwaters for wetland development purposes is limited and has been mostly based on experience from a few prototype sites[1]. Further investigation and evaluation of the use of breakwaters for these purposes is ongoing at WES under the Wetlands Research Program.

Numerical and physical models have also been used as tools to evaluate beach response to detached breakwaters. The shoreline response model GENESIS (GENEralized Model for SImulating Shoreline Change) (Hanson and Kraus 1989b, 1990; Gravens, Kraus, and Hanson 1991) has been increasingly used to examine beach response to detached breakwaters. A limited number of detached breakwater projects have been physically modelled at WES. Good agreement has been obtained in reproducing shoreline change observed in moveable-bed models by means of numerical simulation models of shoreline response to structures (Kraus 1983, Hanson and Kraus 1991).

[1] Personal Communication, 24 February 1993, Dr. Mary Landin, U.S. Army Engineer Waterways Experiment Station, Environmental Laboratory, Vicksburg, MS.

Objectives of Report

A properly designed detached breakwater project can be a viable option for shoreline stabilization and protection at certain coastal sites. The objectives of this report are to summarize and present the most recent functional and structural design guidance available for detached breakwaters, and provide examples of both prototype breakwater projects and the use of available tools to assist in breakwater design.

Chapter 2 presents functional design guidance including a review of existing analytical techniques and design procedures, pre-design site analyses and data requirements, design considerations, and design alternatives. Chapter 3 discusses numerical and physical modeling as tools for prediction of morphological response to detached breakwaters, including a summary of the shoreline response numerical simulation model GENESIS. A summary of moveable-bed physical modeling and modeled breakwater projects is also presented. Chapter 4 summarizes and presents structural design guidance including static and dynamic breakwater stability and methods to determine performance characteristics such as transmission, reflection, and energy dissipation. Other breakwater design issues are discussed in Chapter 5 including beach fill requirements, constructability issues, environmental concerns, and project monitoring. Chapter 6 presents a summary and suggestions for the direction of future research relative to detached breakwater design. Appendix A provides a case example of a breakwater project designed and constructed at Bay Ridge, Maryland, including GENESIS modeling of the project performance. Parameter definitions used throughout the report are given in Appendix B.

2 Functional Design Guidance

Functional Design Objectives

Prototype experience shows that detached breakwaters can be an important alternative for shoreline stabilization in the United States. Shoreline stabilization structures such as breakwaters or groins seek to retain or create a beach area through accretion, as opposed to structures such as seawalls or revetments, which are designed to armor and maintain the shoreline at a specific location. Additionally, breakwaters can provide protection to a project area while allowing longshore transport to move through the area to downdrift beaches.

The primary objectives of a breakwater system are to increase the longevity of a beach fill, provide a wide beach for recreation, and provide protection to upland areas from waves and flooding (EM 1110-2-1617). Breakwaters can also be used with the objective of creating or stabilizing wetland areas. The breakwater design should seek to minimize negative impacts of the structure on downdrift shorelines.

Beach nourishment has become an increasingly popular method of coastal protection. However, for economic and public perception reasons, it is desirable to increase the time interval between renourishments, that is, to lengthen the amount of time that the fill material remains on the beach. This increase in fill longevity can be accomplished through the use of shoreline stabilization structures, such as a detached breakwater system. The combination of beach nourishment and structures can provide a successful means of creating and maintaining a wide protective and recreational beach. Lakeview Park, Ohio, is an example of a recreational beach maintained by a combination of breakwaters, groins, and beach fill (Bender 1992) (Figure 8).

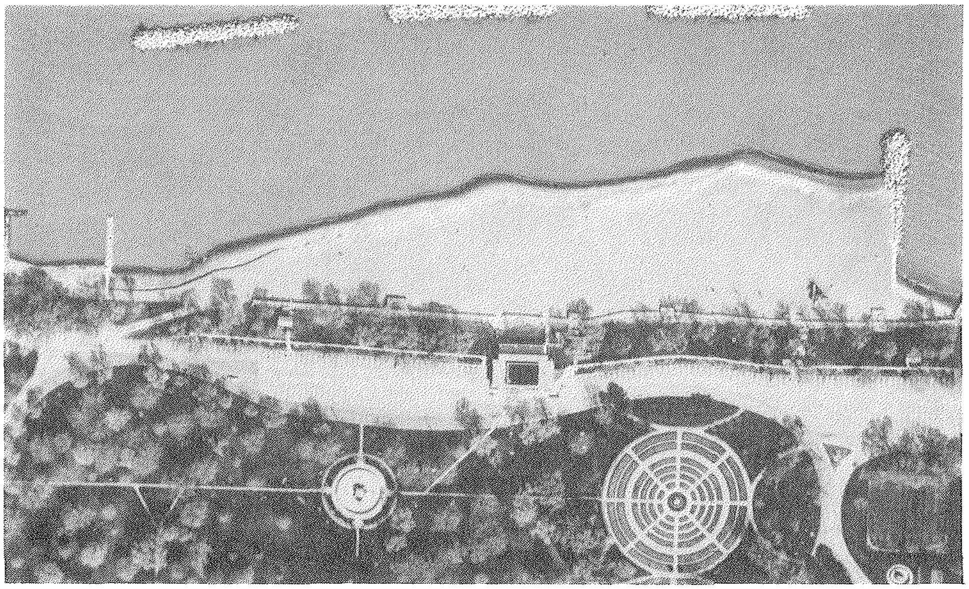


Figure 8. Aerial view of Lakeview Park, Lorain, Ohio

Design of Beach Planform

Types of shoreline configuration

A primary consideration in detached breakwater design is the resulting shoreline configuration due to the structure. Three basic types of beach planforms have been defined for detached breakwaters: tombolo, salient, or limited. A bulge in the shoreline is termed a salient, and if the shoreline connects to the breakwater it is termed a tombolo (see Figure 1). A limited response, or minimal beach planform sinuosity, may occur if an adequate sediment supply is not available or the structure is sited too far offshore to influence shoreline change. Figures 9 to 11 show U.S. prototype examples of each shoreline type.

Selection of functional alternatives

Each planform alternative has different sediment transport patterns and effects on the project area, and certain advantages and disadvantages exist for each. The resulting shoreline configuration depends on a number of factors including the longshore transport environment, sand supply, wave climate, and geometry of the breakwater system.

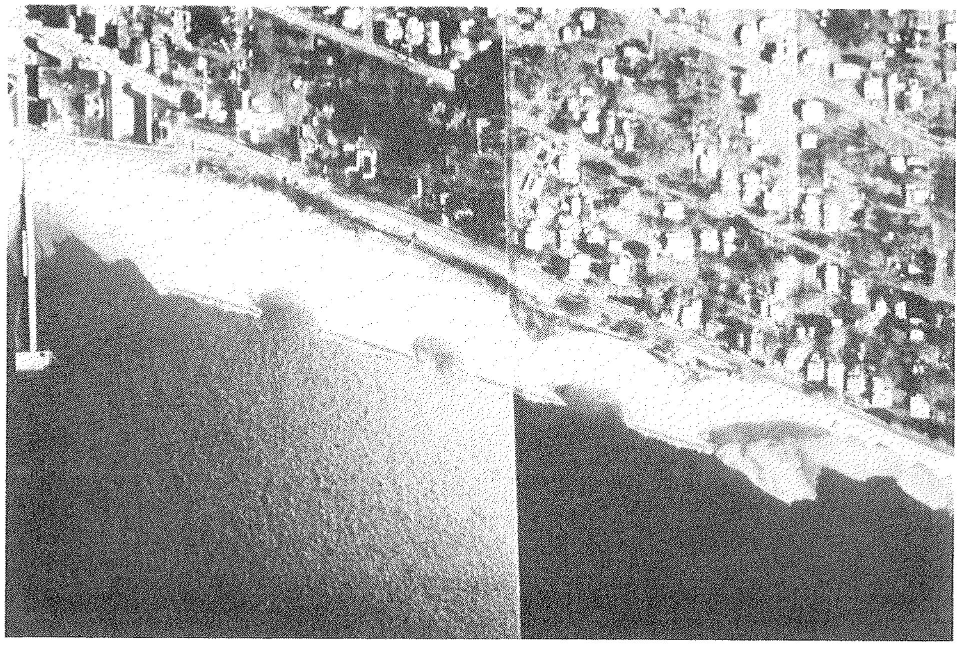
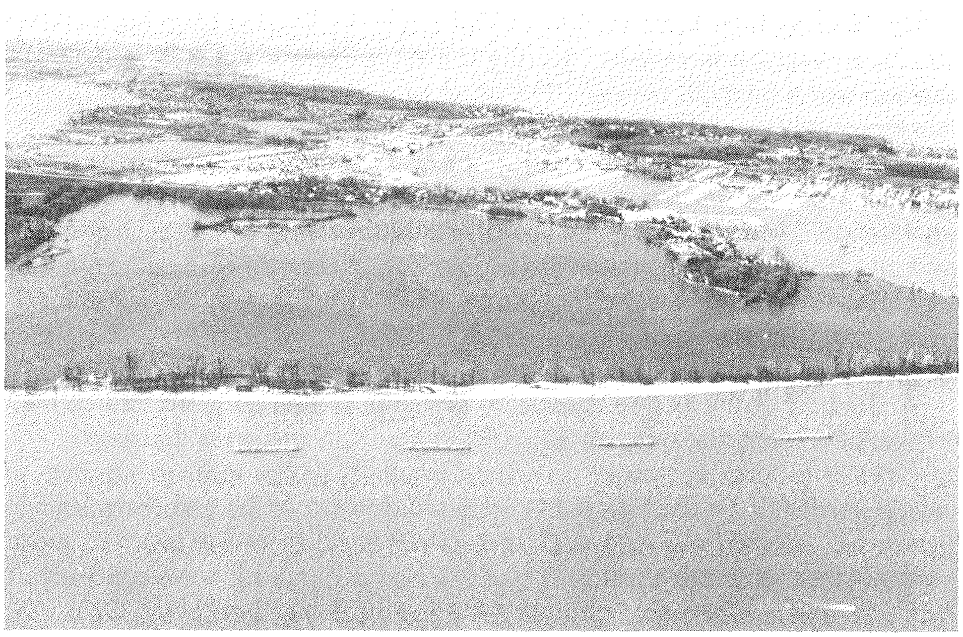


Figure 9. Detached breakwaters with tombolo formations at Central Beach Section, Colonial Beach, Virginia

Figure 10. Salient that formed after initial construction at the Redington Shores, Florida, breakwater

a. Aerial view showing limited response, but bar formation



b. Limited beach response

Figure 11. Limited shoreline response due to detached breakwaters at East Harbor State Park, Ohio

Salient formation. Generally, a salient is the preferred response for a detached breakwater system because longshore transport can continue to move through the project area to downdrift beaches. Salient formation also allows the creation of a low wave energy environment for recreational swimming shoreward of the structure. Salients are likely to predominate if the breakwaters are sufficiently far from shore, short with respect to incident wave length, and/or relatively transmissible (EM 1110-2-1617). Wave action and longshore currents tend to keep the shoreline from connecting to the structure. Pope and Dean (1986) distinguish between well-developed salients, which are characterized by a balanced sediment budget and stable shoreline, and subdued salients, which are less sinuous and uniform through time, and may experience periods of increased loss or gain of sediment.

Tombolo formation. If a breakwater is located close to shore, long with respect to the incident wavelength, and/or sufficiently impermeable to incident waves (low wave transmission), sand will likely accumulate in the structure's lee, forming a tombolo. Although some longshore transport can occur offshore of the breakwater, a tombolo-detached breakwater system can function similar to a T-groin by blocking transport of material shoreward of the structure and promoting offshore sediment losses via rip currents through the gaps. This interruption of the littoral system may starve downdrift beaches of their sediment supply, causing erosion. If wave energy in the lee of the structure is variable, periodic tombolos may occur (Pope and Dean 1986). During high wave energy, tombolos may be severed from the structure, resulting in salients. During low wave energy, sediment again accretes and a tombolo returns. The effect of periodic tombolos is the temporary storage and release of sediment to the downdrift region. If the longshore transport regime in the project area is variable in direction or if adjacent shoreline erosion is not a concern, tombolo formation may be appropriate. Tombolos have the advantages of providing a wide recreational area and facilitated maintenance and monitoring of the structure, although they also allow for public access out to the structure which may be undesirable and potentially dangerous.

Artificial headlands. In contrast to detached breakwaters, where tombolo formation is often discouraged, an artificial headland system is designed specifically to form a tombolo. Artificial headland design seeks to emulate natural headlands by creating stable beaches landward of the gaps between structures. Also termed log-spiral, crenulate-shaped, or pocket beaches, most headland beaches assume a shape related to the predominant wave approach with a curved section of logarithmic spiral form (Chew, Wong, and Chin 1974; Silvester, Tsuchiya, and Shibano 1980). Shoreline configurations associated with headland breakwaters are discussed in Silvester (1976) and Silvester and Hsu (1993). Figure 12 shows the headland breakwater and beach fill system at Maumee Bay State Park, Oregon, Ohio, designed by the USAED, Buffalo (Bender 1992).

Wetland stabilization and creation. Breakwaters can be used as retention or protective structures when restoring, enhancing, or creating wetland areas.

Figure 12. Artificial headland and beach fill system at Maumee Bay State Park, Ohio (from Bender (1992))

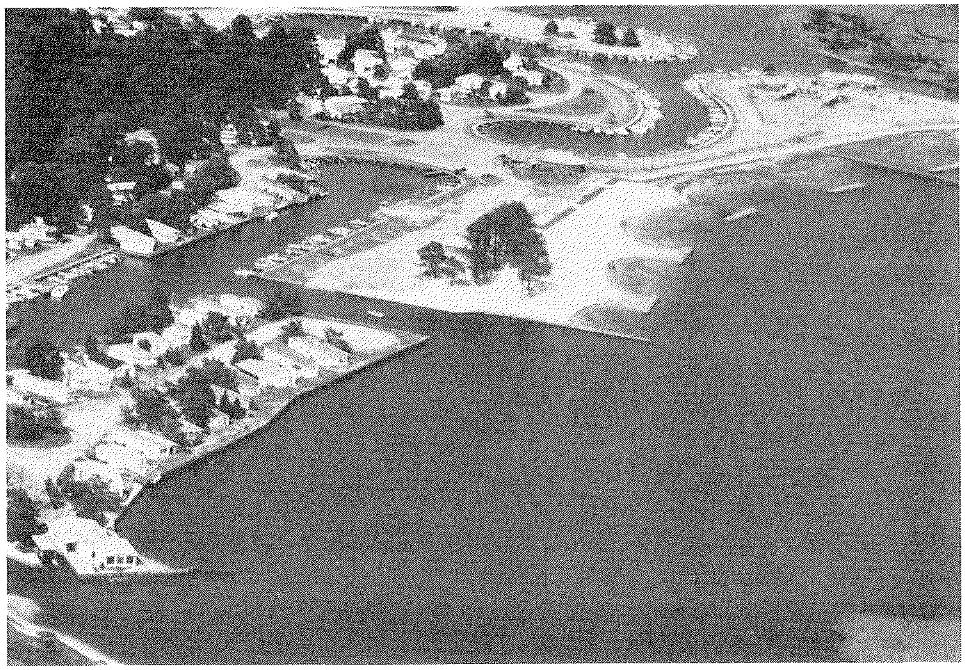
The desired planform behind the breakwater in this type of application is marsh development, the extent of which tends to be site-specific (Figures 13 and 14). The primary objective of the structure is to contain placed dredge material and protect existing or created wetland areas from wave, current, or tidal action. The wetland may or may not extend out to the structure. Depending on the habitat, frequent exchange of fresh or saltwater may be important. Considerations and guidelines for marsh development are provided in EM 1110-2-5026; Knutson, Allen, and Webb (1990); and U.S. Department of Agriculture (1992).

Techniques for controlling shoreline response

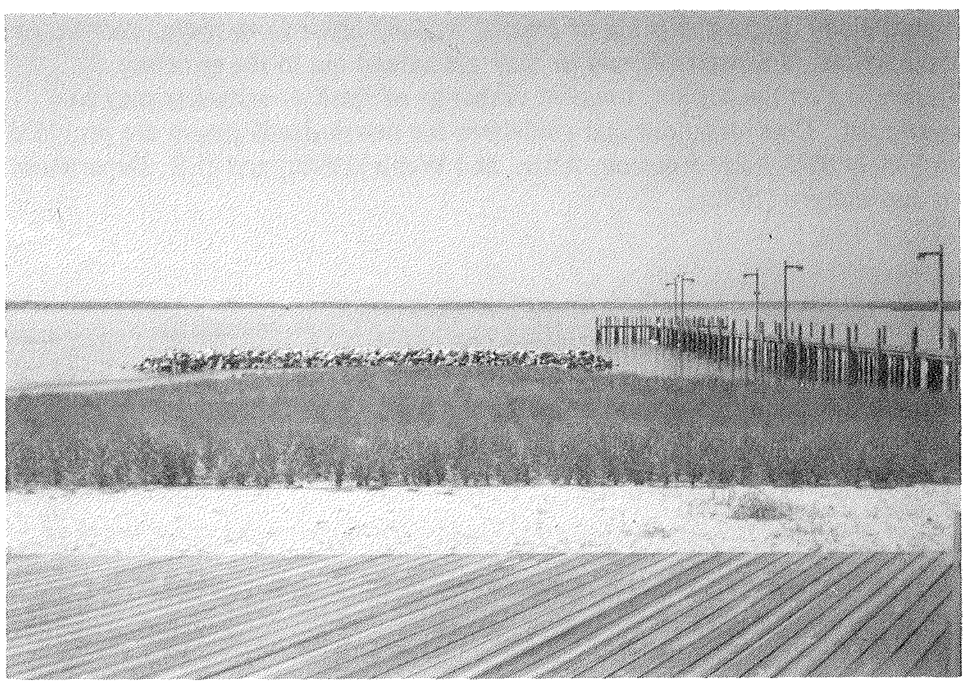
After selection of a desired beach planform, the extent of incident wave reduction or modification to encourage the formation of that planform must be determined. Various techniques and design tools used to predict and control shoreline response are reviewed in later sections of this chapter.

Functional Design Concerns and Parameters

Parameters affecting morphological response and subsequently the functional design of detached breakwaters include wave height, length, period, and angle of wave approach; wave variability parameters such as seasonal changes, water level range, sediment supply and sediment size; and structural parameters such as structure length, gap distance, depth at structure, and



a. Aerial view showing beach and vegetation development



b. Vegetation established in the lee of a breakwater

Figure 13. Pot-Nets breakwater project in Millsboro, Delaware (photos courtesy of Andrews Miller and Associates, Inc.)

Figure 14. Marsh grass (Spartina) plantings behind breakwaters at Eastern Neck, Maryland

structure transmission. Figure 15 provides a definition sketch of parameters related to detached breakwater design. Parameter definitions are provided in Appendix B.

Morphological response characteristics that need to be considered in design are: resultant beach width and planform, magnitude and rate of sediment trapping as related to the longshore transport rate and regional impacts, sinuosity of the beach planform, beach profile slope and uniformity, and stability of the beach regardless of seasonal changes in wave climate, water levels, and storms (Pope and Dean 1986).

Artificial headland design parameters include the approach direction of dominant wave energy, length of individual headlands, distance offshore and location, gap width, crest elevation and width of headlands, and artificial nourishment (Bishop 1982; USAED, Buffalo 1986; Hardaway and Gunn 1991a and 1991b). A definition sketch of an artificial headland breakwater system and beach planform is provided (Figure 16).

Considerations for structures used for wetland development include properties of the dredged material to be retained or protected, maximum height of dredged material above firm bottom, required degree of protection from waves and currents, useful life and permanence of the structure, foundation conditions at the site, and availability of the structure material (EM 1110-2-5026). These considerations will determine whether a structure is feasible and cost-effective at a particular wetland site. If an area is exposed to a high wave energy climate and current action or water depths are too great, a breakwater may not be cost-effective relative to the amount of marsh that will be developed. Although morphological response due to sediment

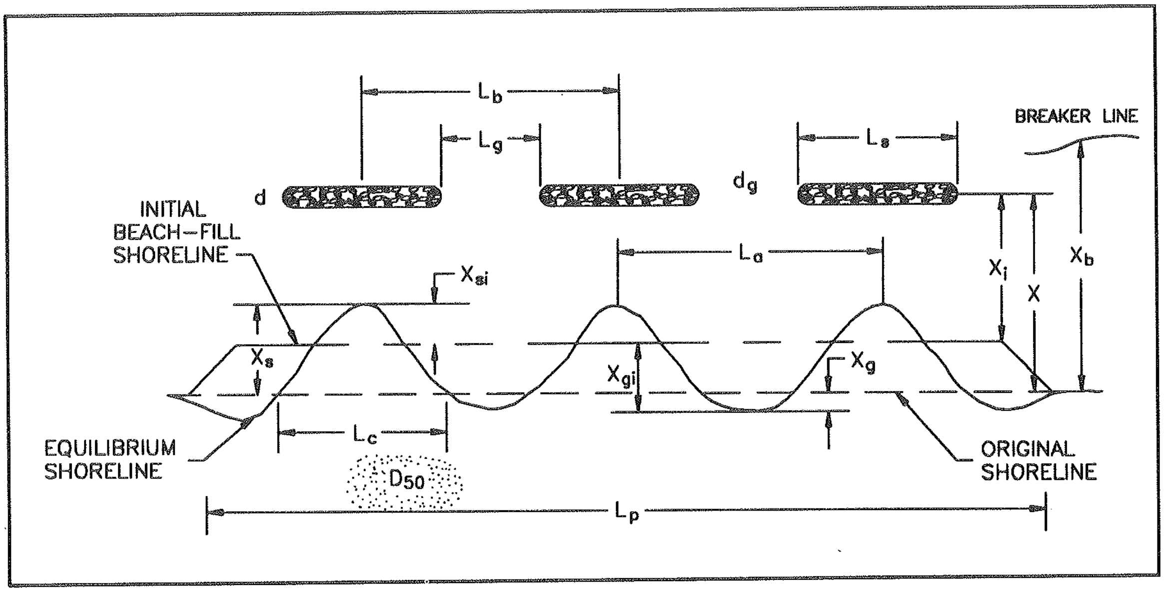


Figure 15. Definition sketch of terms used in detached breakwater design (modified from Rosati (1990)

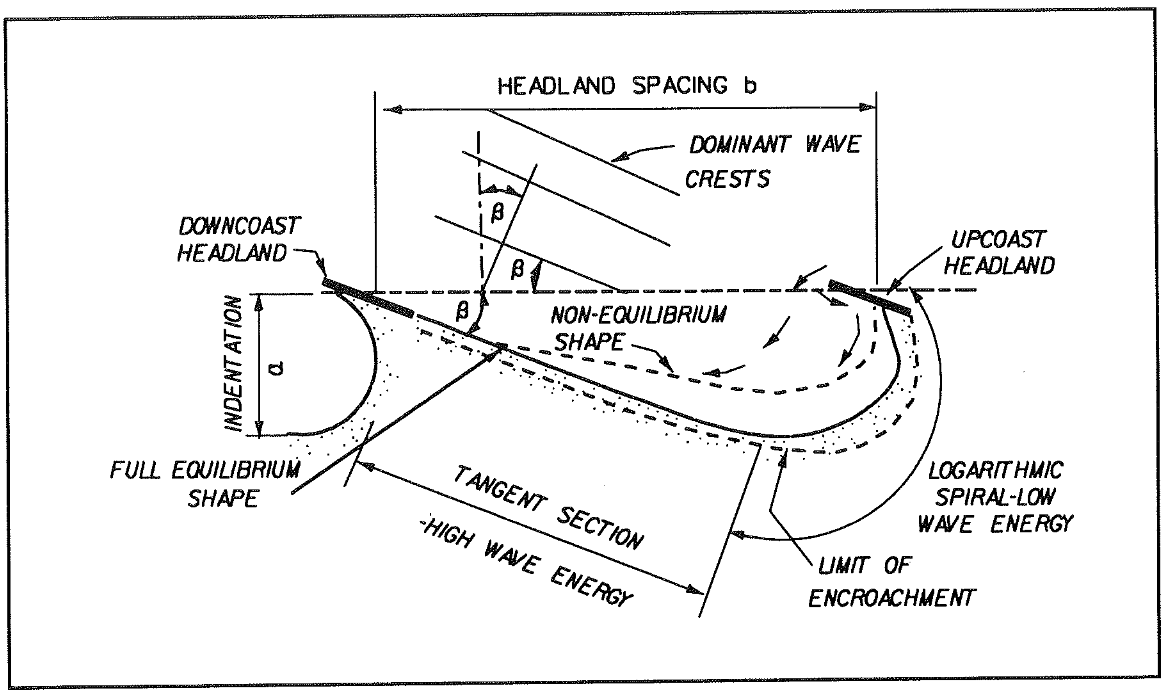


Figure 16. Definition sketch of artificial headland system and beach planform (from EM 1110-2-1617)

transport may not be as significant a concern when using breakwaters for wetlands purposes, many of the design concerns and data requirements, such as wave and current climate, are the same as those necessary for traditional breakwater design. The following sections discuss concerns that must be addressed and evaluated during functional design of a detached breakwater system. The effects of a structure on various coastal processes as well as the effects of coastal parameters on shoreline response are discussed.

Structural considerations

Structural configuration is the extent of protection provided by the structure plan and is defined by several design parameters; segment length, gap width, project length, number of segments, cross-sectional design (transmission), and distance offshore (Pope and Dean 1986). These design parameters should be considered relative to the wave climate and potential effects on coastal processes as described in the following sections.

Single versus multiple segmented system. Use of single offshore breakwaters in the United States is not a new concept; however, most have been built with the objective of providing safe navigation and not as shore protection or stabilization devices. One of the first single rubble-mound breakwater projects was constructed at Venice, California, in 1905 for the initial purpose of protecting an amusement pier. A tombolo eventually formed in the lee of the Venice breakwater (Figure 17). Use of segmented systems in the United States has been limited in general, but has increased substantially in the past two decades (for example, see Figures 2, 7, 8, and 18). The use of segmented systems as shore protection devices has been more extensive in other countries such as Japan, Israel, and Singapore (see Figures 3 and 4) than in the United States.

The decision to use a single versus a multiple system is essentially based on the length of shoreline to be protected. If a relatively long length of shoreline needs to be protected and tombolo development is not desired, a multiple segmented system with gaps should be designed. Construction of a single long breakwater will result in the formation of a single or double tombolo configuration. As discussed previously, tombolo formation in a continuous littoral system may adversely impact downdrift beaches by blocking their sediment supply. A properly designed multiple system will promote the formation of salients, but will continue to allow a percentage of the longshore transport to pass through the project area, thus minimizing erosion along the downdrift shorelines.

The number of breakwaters, their length, and gap width are dependent on the wave climate and desired beach planform. Several long breakwaters with wide gaps will result in a sinuous shoreline with large amplitude salients and a spatial periodicity equal to the spacing of the structures; that is, there will be a

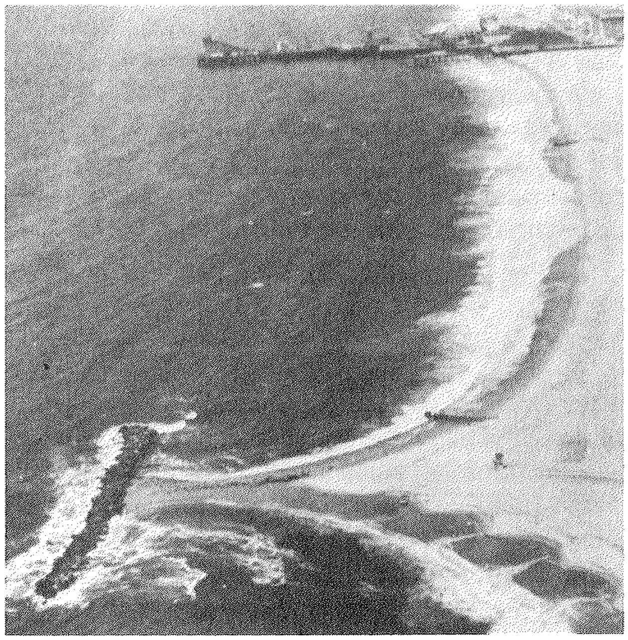


Figure 17. Single detached breakwater at Venice Beach, California

Figure 18. Segmented detached breakwaters near Peveto Beach, Louisiana

large salient behind each breakwater (EM 1110-2-1617) (Figure 19a). Numerous more closely spaced segments will also result in a sinuous shoreline, but with more closely spaced, smaller salients (Figure 19b). If uniform shoreline advance is desired, a segmented system with small gaps or a single long breakwater with adequate wave overtopping and transmission should be considered.

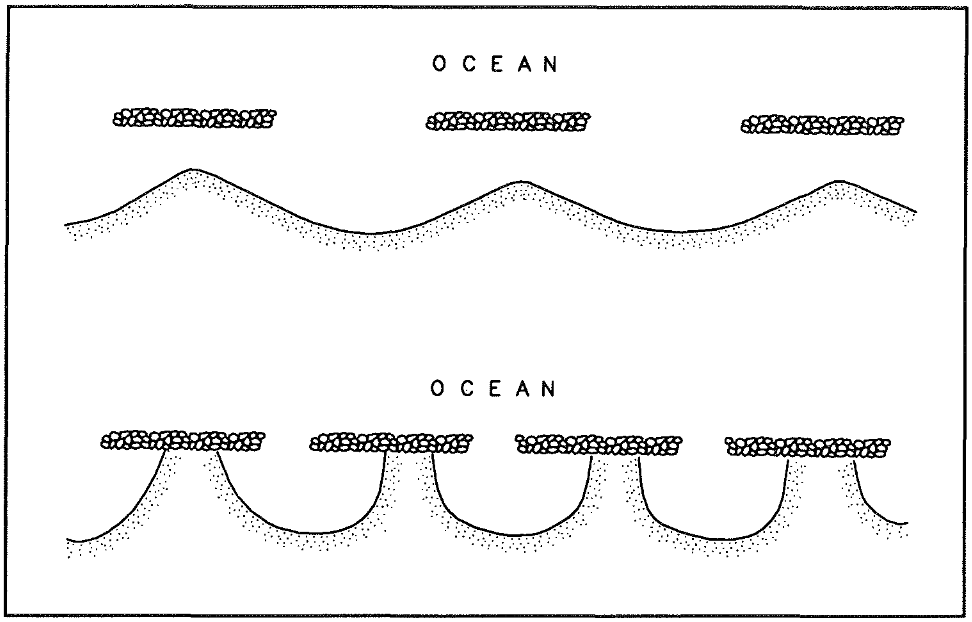
Gap width. Wide gaps in a segment system allow more wave energy to enter the area behind the breakwaters. The ratio of gap width to wave length can significantly affect the distribution of wave height in the lee (Dally and Pope 1986). By increasing the gap-to-wave length ratio, the amount of wave energy penetrating landward of the breakwaters is increased.

Wave diffraction at a gap can be computed using the numerical shoreline response model GENESIS (Hanson and Kraus 1989b, 1990; Gravens, Kraus, and Hanson 1991). GENESIS calculates diffraction and refraction for random waves and accounts for wave shoaling and breaking. The effect of diffraction on a wave which passes through a gap can also be calculated using diffraction diagrams found in the *Shore Protection Manual* (SPM) (1984); however, these simple diagrams are for monochromatic waves and do not account for wave shoaling or breaking. If the design wave breaks before passing the breakwater, values estimated by the diagrams could be significantly higher than may be expected.

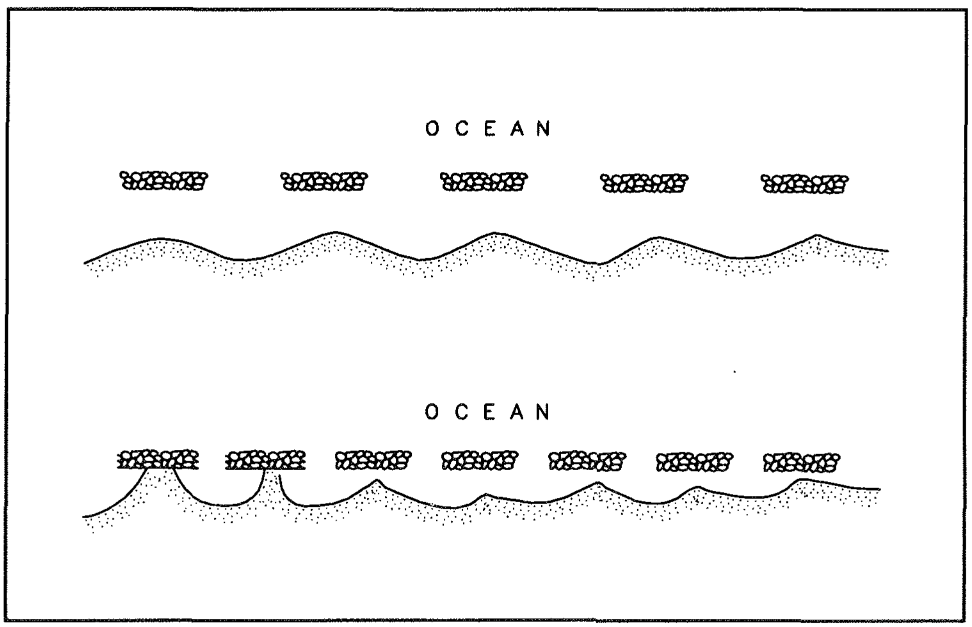
Dally and Pope (1986) suggest that gaps should be sized according to the desired equilibrium shoreline position opposite each gap. Unless the gap-to-incident wave length ratio is very small, there will be minimal reduction in wave height at the shoreline directly opposite each gap. Without an adequate sediment supply, the shoreline will probably not accrete and may even erode in these areas. Generally, Dally and Pope recommend that gaps should be at least two wave lengths wide relative to those waves that cause average sediment transport.

The "exposure ratio" is defined as the ratio of gap width to the sum of breakwater length and gap width, or the fraction of the shoreline directly open to waves through the gaps (EM 1110-2-1617). Exposure ratio values for various prototype projects are provided in Table 2 and range from 0.25 to 0.66. Projects that are designed to contain a beach fill within fixed boundaries have larger ratios (such as Presque Isle, Pennsylvania). Comparatively, the ratio at Winthrop Beach, Massachusetts, where wide gaps were included to allow for small craft navigation, is 0.25. Comparison of these prototype values provides insight to project design at other locations.

Structure orientation. The size and shape of the resulting planform can be affected by the breakwater's orientation relative to incident wave angle and orientation of the pre-project shoreline. Shoreline configuration will change relative to the wave diffraction patterns of the incident waves. If incident wave energy is predominantly oblique to the shoreline, orientation of the



a. With a few relatively long, widely spaced segments



b. With more numerous, shorter, closely spaced segments

Figure 19. A segmented breakwater system (from EM 1110-2-1617)

Table 2
"Exposure Ratios" for Various Prototype Multiple Breakwater Projects[1] (Modified from EM 1110-2-1617)

Project	Exposure Ratio	Shoreline Response
Winthrop Beach, MA	0.25	Permanent tombolos (low tide); well-developed salients (high tide)
Lakeview Park, Lorain, OH	0.36	Subdued salients
Castlewood Park, Colonial Beach, VA	0.31 to 0.38	Permanent tombolos
Central Beach, Colonial Beach, VA	0.39 to 0.45	Periodic tombolos
East Harbor, State Park, OH	0.56	Limited
Presque Isle, Erie, PA (experimental prototype) (hydraulic model)	 0.56 to 0.66 0.60	Permanent tombolos

[1] The "exposure ratio" is defined as the ratio of gap width to the sum of the breakwater length and gap width. It is the fraction of shoreline directly exposed to waves and is equal to the fraction of incident wave energy reaching the shoreline through the gaps. A "sheltering ratio" that is the fraction of incident wave energy intercepted by the breakwaters and kept from the shoreline can also be defined. It is equal to 1 minus the "exposure ratio."

breakwater parallel to incoming wave crests will protect a greater length of shoreline and reduce toe scour at the breakwater ends.

Location with respect to breaker zone. If the breakwater is placed substantially landward of the breaker zone, tombolo development may occur. However, a significant amount of longshore transport may continue to pass seaward of the breakwater, thus alleviating the effects of a tombolo on downdrift shorelines. A disadvantage of a breakwater within the breaker zone may be substantial scour at the structure's toe. Generally, detached breakwaters designed for shore protection along an open coast are placed in a range of water depths between 1 and 8 m (Dally and Pope 1986).

Structural mitigation methods for impacts on adjacent shorelines. End effects from a breakwater project can be reduced by creating a gradual transition or interface between the protected shoreline and adjacent shorelines (Hardaway, Gunn, and Reynolds 1993). Hardaway, Gunn, and Reynolds (1993) document various methods for structurally transitioning the ends of breakwater systems in the Chesapeake Bay. Structural methods used at the 12 sites investigated include shorter and lower breakwaters, hooked or inclined groins, small T-head groins, and spur-breakwaters. Based on project experience in the Chesapeake Bay, Hardaway, Gunn, and Reynolds (1993) recommend hooked or skewed groins where adjacent effects are predicted to be minimal; T-head groins where the dominant direction of wave approach is shore-normal; and short groins, spur-breakwaters and low breakwaters placed close to shore when the dominant wave direction is oblique. The use and design of these methods will vary with each breakwater project site. If possible, shoreline morphology, such as a natural headland or creek, should be used to terminate the breakwater project and minimize impacts on adjacent shorelines.

Wave climate

Structural effects on wave environment. Breakwaters reduce wave energy at the shoreline by protecting the shoreline from direct wave attack and transforming the incoming waves. Wave energy is dissipated on and reflected from the structure, or diffracted around the breakwater's ends causing the waves to spread laterally. Some wave energy can reach the breakwater's lee by transmission through the structure, regeneration in the lee by overtopping waves, or diffraction around the structure's ends. As most detached breakwater projects are constructed in shallow water, incident wave energy is often controlled by local water depth and variability in nearshore bathymetry. Average wave conditions, as opposed to extreme or storm wave conditions, generally control the characteristic condition of the shoreline.

Wave diffraction. Shoreline response to detached breakwaters is primarily controlled by wave diffraction. The diffraction pattern and wave heights in the breakwater's lee are determined by wave height, length, and angle, cross-sectional design, and for segmented structures, the gap-to-wave length ratio. The resulting shoreline alignment is generally parallel to the diffracted wave crests.

If incident breaking wave crests are parallel to the initial shoreline (a condition of no longshore transport), the waves diffracted into the breakwater's shadow zone will transport sediment from the edges of this region into the shadow zone (Fulford 1985). This process will continue until the beach planform is parallel to the diffracted wave crests and zero longshore transport again results (Figure 20). For oblique incident waves, the longshore transport rate in the breakwater's lee will initially decrease, resulting in sediment deposition (Figure 21). A bulge in the shoreline will develop and continue to grow until a new equilibrium longshore transport rate is restored or a tombolo results.

Wave height. The magnitude of local diffracted wave heights is generally determined by their distance from the breakwater's ends, or by their location relative to the gaps in a segmented system (EM 1110-2-1617). Wave height affects the pattern of diffracted wave crests, and therefore affects the resulting beach planform. For shallow water of constant depth, linear wave theory predicts the circular pattern of diffracted wave crests shown in Figure 22a. However, for very shallow water where wave amplitude affects wave celerity C, the celerity decreases along the diffracted wave crests in relation to the decrease in wave height. Figure 22b shows the distorted diffraction pattern, a series of arcs of decreasing radius, which results. The latter situation usually results in tombolo formation if the undiffracted portion of the wave near the

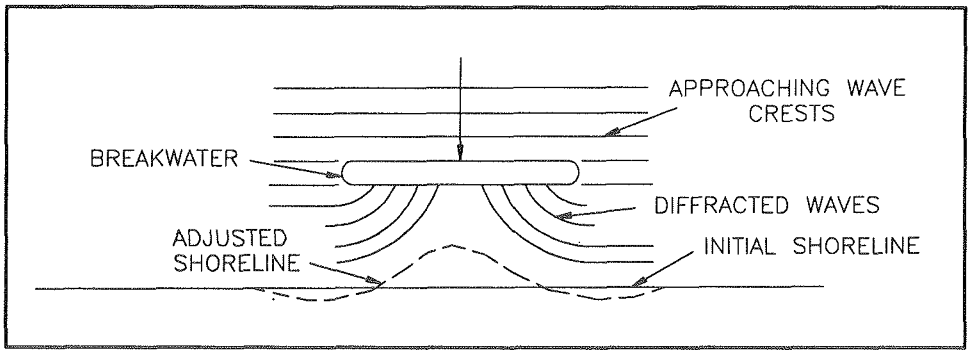


Figure 20. Shoreline response due to wave crests approaching parallel to the shoreline (from Fulford (1985))

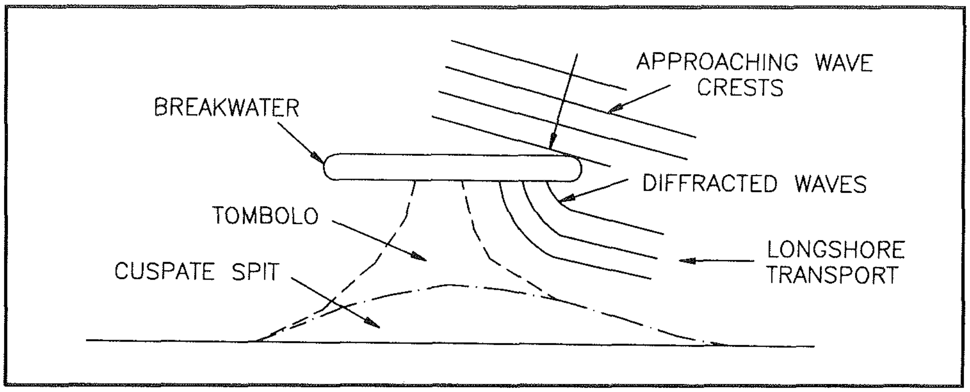
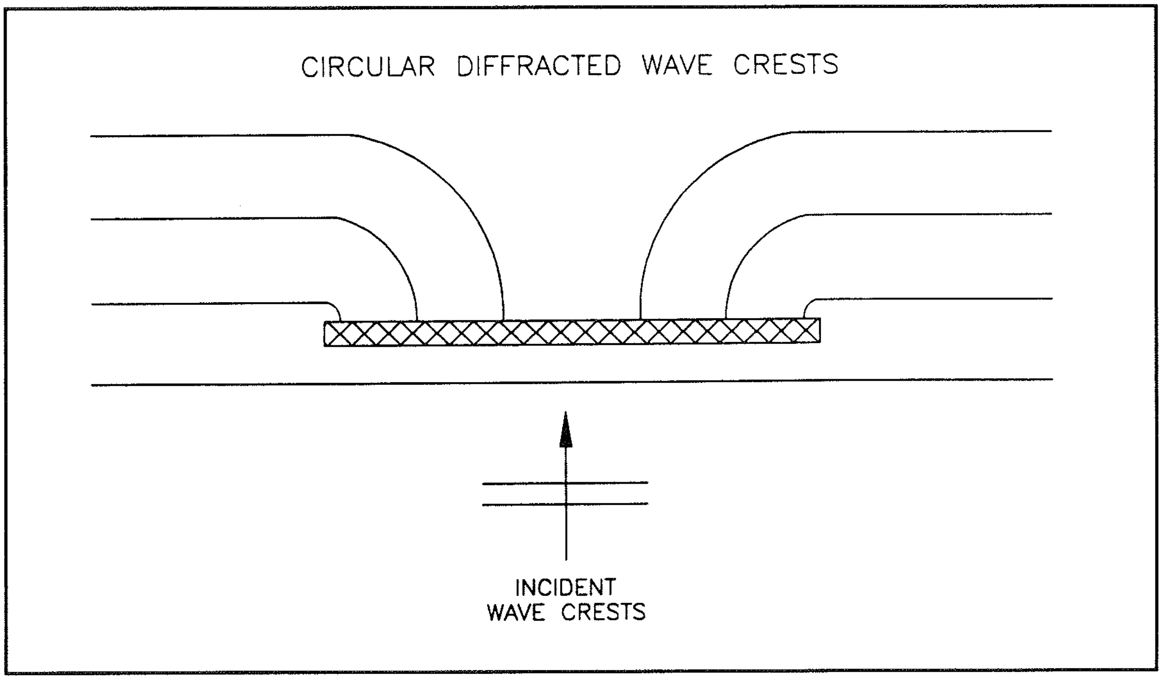


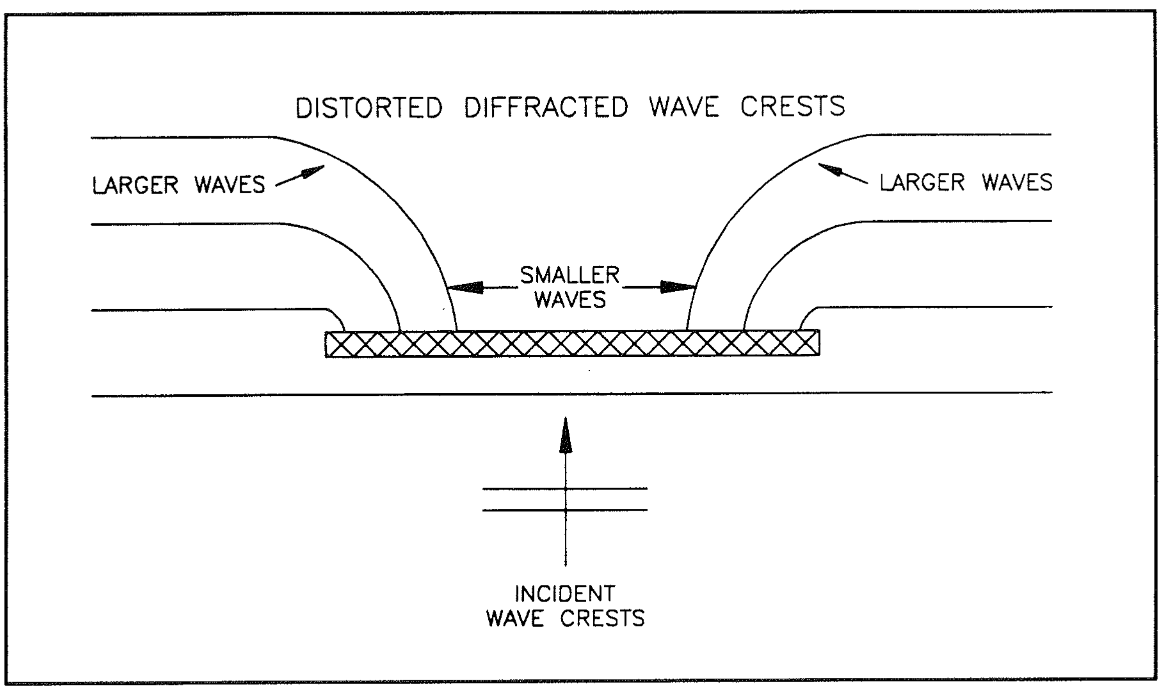
Figure 21. Shoreline response due to wave crests approaching obliquely to the shoreline (from Fulford (1985))

breakwater reaches the shore before the waves diffracted around the structure's ends intersect (Dally and Pope 1986).

Wave overtopping and transmission. Wave energy transmitted landward of the breakwater due to overtopping and transmission through the structure can also affect beach planform development and stability. If adequate wave energy is allowed to pass through or over the structure, tombolo formation can be prevented and/or salient formation can be inhibited. Tide level, wave height and period, and structure slope and roughness all have effects on the amount and form of energy transmitted due to overtopping (*Shore Protection Manual* 1984). If overtopping occurs, the beach planform tends to flatten and spread laterally in a uniform manner; however, waves overtopping the structure have a shorter period than the incident wave and are highly irregular. Wave energy passing through the structure is transmitted at the same period as the incident waves, and is often more predictable and regular than that produced by overtopping. In design, wave heights due to overtopping are generally determined by the structure's crest elevation, and wave transmission through a breakwater is determined by the structure's permeability. A low-crested reef type breakwater is designed to allow periodic overtopping of the structure by incident waves, thus preventing



a. Diffraction at a breakwater assuming linear wave theory



b. Diffraction at a breakwater including the effects of amplitude and dispersion

Figure 22. Comparison of diffraction pattern theory (from Dally and Pope (1986))

tombolo formation. Wave transmission is discussed in more detail in Chapter 4, *Structural Design Guidance*.

Wavelength. Generally, the amount of wave energy diffracted into a structure's lee increases with increasing wavelength. Assuming monochromatic waves and a flat bottom, wave length will not change the pattern made by the wave crests, but will affect the wave height at each location. An analysis using the diffraction diagrams provided in the *Shore Protection Manual* (1984) can simplistically compute the amount of energy that reaches the lee of the breakwater. An example problem using the diffraction analysis is presented in Dally and Pope (1986).

Wave angle. Equilibrium beach planform and degree of salient development can be significantly affected by incident wave angle relative to both the shoreline and structure. Design must not only consider predominant wave direction, but also the average annual wave angle distribution. Salients and tombolos tend to align with the predominant wave direction. Generally, the feature's apex is near the center of the breakwater and is filled more on the updrift than the downdrift side. If predominant waves are extremely oblique to the shoreline, the beach planform and feature's apex can be shifted downdrift and can change with seasonal variations in wave direction. Oblique waves can also drive a regional longshore current, which may dominate local effects of the breakwater and limit salient development. Increasing the structure's length can subdue the effect of the oblique waves.

Wave conditions seaward of breakwater. Waves reflected from the seaward side of the structure can sometimes interact with incident waves and cause a partial standing wave pattern seaward of the breakwater (EM 1110-2-1617). This increased wave action can cause scour on the seaward side of the structure, potentially creating foundation problems. A structure's reflectivity is largely determined by crest elevation, permeability, and type of construction material. Rubble-mound structures are the least reflective detached breakwater construction type.

Effects of breakwater on nearshore currents

Construction of a breakwater system can affect nearshore currents in two ways: reduction of longshore current in the vicinity of the structure, and creation of a net seaward flow of water through gaps in a segmented system (EM 1110-2-1617). On an open-coast beach, a longshore current is generated by waves approaching the shoreline at an angle. The placement of a structure introduces an interruption to this natural system. The longshore current will generally respond by slowing or stopping when it moves into the project area, thus reducing the current's sediment carrying capacity and depositing sand in the structure's lee. The structure's length and distance from shore are two design parameters that must be considered when evaluating the breakwater's effect on longshore currents and sediment transport. For example, a relatively long breakwater will cause a greater reduction of longshore current in the project area than a short breakwater.

If the breakwater's crest elevation is sufficiently low and overtopping occurs, water level behind the breakwater is increased and flow occurs around the structure. In a multiple segment system, this results in a net seaward flow through the gaps, which can cause offshore sediment losses, structural scour, and create a hazard to swimmers. The magnitude of return currents through the gaps can be reduced by increasing crest elevation, gap width, and/or structure permeability. Seelig and Walton (1980) present a method for estimating flow rate through the gaps of offshore segmented breakwaters caused by wave overtopping. The effects of wave height and period, breakwater freeboard, breakwater length and spacing, distance offshore, water depth, and shore attachment are considered relative to flow rate through the gaps. Seelig and Walton (1980) recommend that the gap velocity should not exceed 0.5 ft/sec (0.15 m/sec) for extreme design conditions. Velocities greater than this could cause significant offshore losses of sediment and scour around the structure's foundation.

Effects of breakwater on longshore transport

The longshore transport rate Q is the rate at which littoral material moves alongshore in the surf zone from currents produced by breaking waves. Detached breakwaters can significantly reduce longshore transport through a project area. Reduction of wave heights and wave diffraction around the breakwater's ends primarily determines the reduction in transport capacity. If a salient forms, longshore transport can continue to move through the project area; however, a tombolo can act as a total barrier of longshore transport causing a sediment deficiency at downdrift beaches. Some longshore transport may be redirected seaward of the breakwater, but may also result in an offshore loss of material. Structure length, distance offshore, crest elevation, and gap width may be modified to vary the resulting transport rate during design of a breakwater system. Once constructed, modifications to the transport rate are more difficult; however, reduction of crest elevation or increasing permeability can be undertaken to allow more wave energy to penetrate the structure. This was conducted at the Redington Shores, Florida, detached breakwater project where tombolo formation and subsequent blocking of longshore transport occurred (Chu and Martin 1992).

The effects of a breakwater on the shoreline depend on both net and gross transport rates. Shoreline response both at the structure and on adjacent shorelines can occur rapidly if transport rates are large, or can take several years for low transport rates. If net transport in a project area is nearly zero, but gross transport is not zero, the breakwater's major effects will be limited to the general vicinity of the structure; however, some effects of the structure can be experienced on updrift and downdrift beaches over time.

Effects of breakwater on onshore-offshore transport

Breakwater construction can reduce offshore transport by presenting a physical barrier to offshore transport and by reducing wave heights and wave steepness, which tends to promote onshore transport of material in the breakwater's lee. However, for segmented systems, especially low-crested, impermeable structures, a net seaward return flow of water can occur through the gaps, promoting offshore loss of sediment. Reduction of seaward flow through the gaps was discussed in the previous section.

Influence of other coastal parameters

Water levels. Water level variations influence the magnitude of wave energy in the lee of the breakwater, which in turn influences shoreline configuration and consequently must be considered in functional design. Dally and Pope (1986) suggest that water level fluctuations of over 1.5 m will tend to hinder permanent tombolo formation, especially if significant wave overtopping of the structure occurs, and may prevent the salient from attaining a smooth equilibrium shape. The Winthrop Beach, Massachusetts, project experiences a relatively large tidal range (2.7 m) and has two distinct planforms during high and low tide conditions (Figure 23). Projects constructed on the Great Lakes or Chesapeake Bay will experience less dramatic water level fluctuations; however, variations in water level may cause significant seasonal or longer period changes in the equilibrium beach planform.

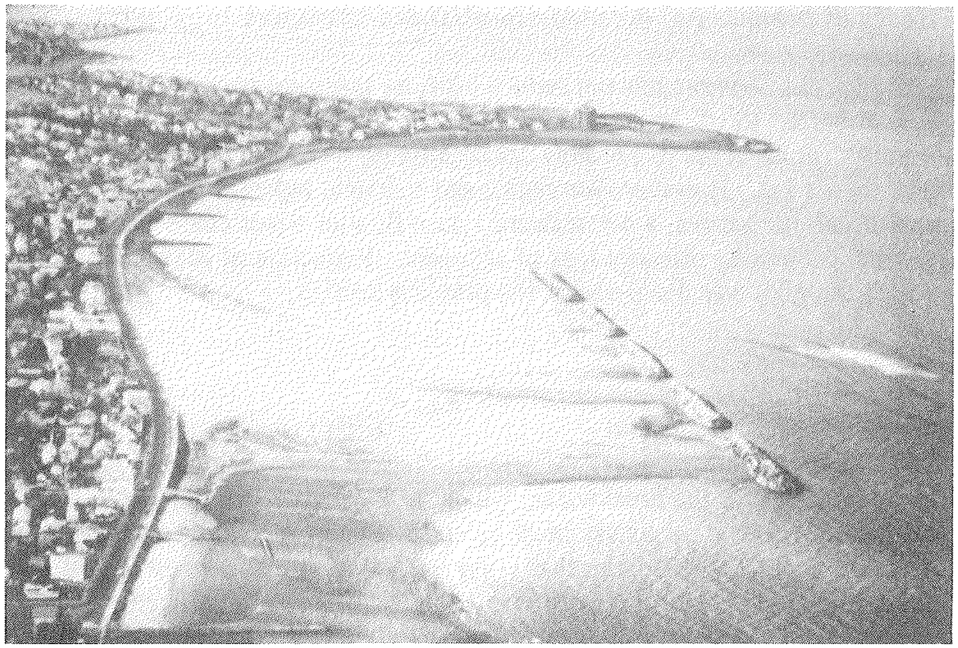
Sediment characteristics. Sediment particle size and distribution affect longshore transport and profile shape, and therefore have some influence over the resulting beach planform. Because a coarse-grained beach equilibrium profile will be steeper, a structure should be placed in relatively deeper water (Dally and Pope 1986).

Data Requirements for Design

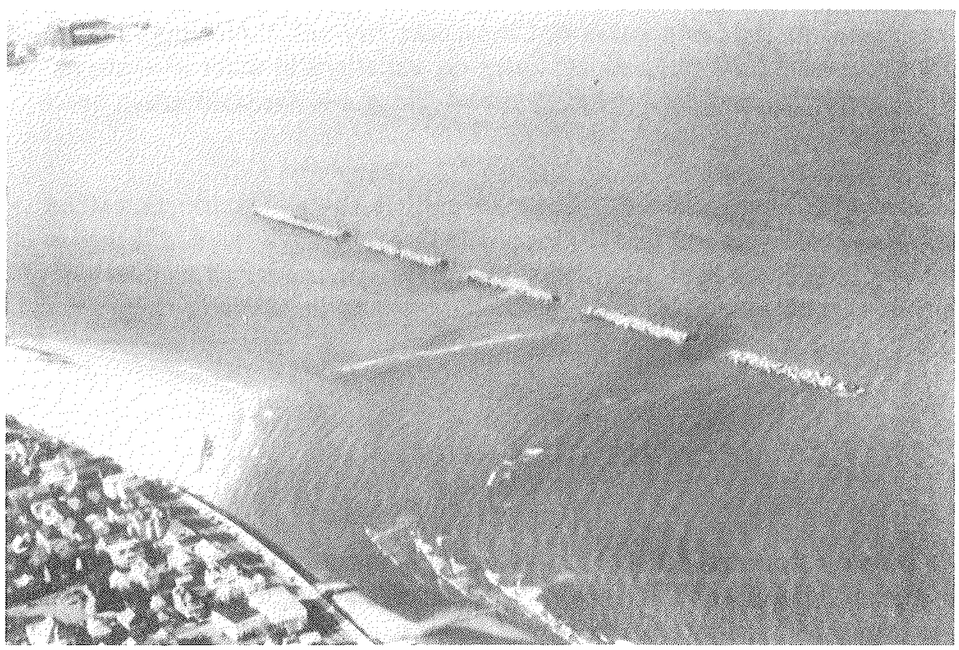
Data requirements for both functional and structural design depend on the methods and evaluation tools used in the specific project design. This section discusses data requirements necessary for an understanding of site characteristics and coastal processes relative to functional design of detached breakwaters.

Water levels

Both the functional and structural design of detached breakwaters require data on the range of water levels that can be expected to occur at a project site. Prevailing water levels will determine where waves may affect the beach



a. Low tide conditions showing periodic tombolo formations



b. High tide conditions showing salient formations with tombolos submerged

Figure 23. Breakwater at Winthrop Beach, Massachusetts, in 1981 (from Dally and Pope (1986))

profile and where wave forces may act on a structure (EM 1110-2-1617). The use of water levels in structural design is described in Chapter 4.

Water level variations are caused by astronomical tides, storm tides, and for the Great Lakes, long-period hydrologic factors and seiches. Design water levels are usually described statistically in terms of the frequency, or probability that a given water level will be equaled or exceeded, or its return period in years. The design may also include storm surge with a specified return period, and/or may account for increased water levels due to sea level rise. Detailed information on the prediction of tides and storm surges is provided in EM 1110-2-1414, *Water Levels and Wave Heights for Coastal Engineering Design*, and EM 1110-2-1412, *Storm Surge Analysis and Design Water Level Determination*.

As described in EM 1110-2-1617, water level data for coastal areas are available from the National Oceanic and Atmospheric Administration's (NOAA) National Ocean Service (NOS) for areas where NOAA operates tide gauges. Tide tables containing water level information are published annually by NOAA. Data on historical water levels of the Great Lakes are available from NOS and from sources such as the USAED, Detroit (for example, USAED, Detroit (1986)), which provides monthly summaries of actual and predicted lake levels. Other sources of water level data include USACE General Design Memoranda for specific project sites and Federal Emergency Management Agency flood insurance studies. Water level statistics for the east coast are presented in Ebersole (1982), and in Harris (1981) for predicted astronomical tides.

Waves

Wave data are required for both the functional and structural design of detached breakwaters. Structural design generally focusses on larger waves in the wave climate, whereas functional design examines a complete data set and includes smaller waves that can cause sediment transport. Data requirements for structural design are discussed in Chapter 4.

Waves primarily control beach planform development at a breakwater project since they contribute to both cross-shore and longshore sediment transport. For functional design, time series of wave height, period, and direction are needed for determination of longshore transport rates in the vicinity of the project. Incident wave heights, periods, and direction are also used to determine wave conditions in the lee of the breakwater and to estimate the resulting beach planform. The average, extremes, and seasonal variability of the waves define the energy available for sediment transport. However, the equilibrium beach planform is generally determined by the average range of conditions rather than extreme events. The prevailing wave direction will generally determine shoreline orientation as the shoreline aligns itself parallel with the wave crests (see Figure 21). If wave direction changes and persists over some time (for instance seasonally), the shoreline will begin to shift in response to the new approach direction.

The two primary forms of wave data for breakwater design are hindcast data and wave gauge data. Wave height statistics to determine design conditions will generally be based on hindcast data since a relatively long record is needed for data extrapolation. The Wave Information Study (WIS) conducted by the USACE has developed hindcast data for all three ocean coasts and the Great Lakes (Jensen 1983; Hubertz et al. 1993; Jensen et al. 1992). EM 1110-2-1414 and EM 1110-2-1502 provide extensive lists and contacts on ways to obtain meteorological and oceanographic data, as well as sources of WIS data and information.

Longshore sand transport rates

Longshore transport of littoral material is the most significant process determining beach planform response to breakwaters. Transport rates are needed to determine what type of planform will develop, sediment budget calculations, beach fill requirements, and potential effects of a project on downdrift beaches.

Longshore transport is typically described in terms of annual net and gross transport rates (*Shore Protection Manual* 1984, EM 1110-2-1617, EM 1110-2-1502). To an observer looking seaward, transport can be to the right Q_R or to the left Q_L, with Q_R being a positive quantity and Q_L assigned a negative value. The annual net transport rate is the net amount of sediment moving past a point on the beach in a year with direction considered and can be computed as:

$$Q_N = Q_R + Q_L \tag{1}$$

The annual gross transport rate is the total amount of sediment moving past a point, regardless of direction, defined as:

$$Q_G = |Q_R| + |Q_L| \tag{2}$$

It is possible that Q_N and Q_G could have substantially different magnitudes, i.e., a large gross transport may exist for a project area, but net transport could be close to zero. The net transport rate is often used to examine erosion rates on adjacent beaches at breakwater or other coastal structures.

Estimates of left, right, net, and gross transport rates can be calculated from wave data that include wave heights, periods, and directions. Usually, determination of the net and gross transport rates will be adequate; however, a time series of transport rates can be calculated if a wave time series is available. The *Shore Protection Manual* (1984) suggests four ways of computing longshore transport rates at a project site. Method 1 involves adopting a transport rate from a nearby site; Method 2 entails calculation of volume change over a specific time period at a known feature such as an inlet or coastal structure; Method 3 depends on the longshore component of energy flux in the surf zone to compute a potential longshore transport rate, also known as the CERC formula; and Method 4 provides an empirical estimate of Q_G. These methods are discussed in detail in the *Shore Protection Manual* (1984), EM 1110-2-1617, and EM 1110-2-1502. Another method commonly used along the Great Lakes and Pacific coast develops a sediment budget based on estimates of inputs including bluff recession and stream sediment contributions. The Shoreline Modeling System (Gravens 1992) includes programs to calculate Q_L, Q_R, and Q_G for a given time series.

It is important to examine longshore transport variability as part of functional design, since transport rates can fluctuate significantly on a monthly, seasonal, or yearly basis. Beach planform can vary and shift substantially in response to prevailing transport conditions.

Offshore bathymetry

Offshore bathymetry is required to aid in determining the distance offshore at which the breakwater will be constructed, wave and current forces which the structure will be subjected to, and quantities of construction materials. Knowledge of offshore bathymetry is also needed to examine wave transformations that may affect the local wave environment at the site. Additionally, bathymetry and beach profiles can provide data to determine the closure depth (the depth beyond which there is no significant sediment transport), if they extend to a sufficient depth and have sufficient vertical and horizontal control to allow comparison of profiles. Depth of closure can also be estimated by reference to a maximum seasonal or annual wave height (Hanson and Kraus 1989b, Hallermeier 1983).

Bathymetric surveys of the project vicinity during the planning and design stages should be conducted for detailed site data. Less accurate bathymetry information can be acquired from U.S. Geological Survey quadrangle topographic maps and or Naval Hydrographic Office charts; however, bathymetry is continually changing and these sources generally do not maintain the most up-to-date information.

Shoreline change

Shoreline change data are required primarily to determine short- and long-term erosion and accretion rates at a project site, prior to design of a breakwater system. This information is necessary to determine the breakwater's location relative to the post-project shoreline and to estimate the volume of sand that will accumulate behind the breakwater.

Historical and recent shoreline change data include beach profile surveys, aerial photography, and other records documenting changes in the shoreline configuration such as beach nourishment data. During the planning and design stages of a project, beach profile data and aerial photography should be acquired to provide an improved understanding of the nearshore system.

Sediment budget

A sediment budget is a quantitative balance of the sources (gains) and sinks (losses) within a project area (*Shore Protection Manual* 1984; EM 1110-2-1502). Sources of sediment include longshore transport, cross-shore transport, aeolian or wind-blown transport, bluff recession, stream or river sediments, and beach fill material. Losses of material to the system may include longshore transport, offshore transport, aeolian transport, offshore canyons, trapping by tidal inlets, blocking by structures, and dredging operations. Generally, a sediment budget is developed for pre-project conditions and then the effects of project construction can be evaluated by making various assumptions regarding the project's effects on transport (EM 1110-2-1617).

Geotechnical data

The physical properties of underlying soils should be investigated and characterized by the collection of soil borings. In the coastal zone, beach sands are often underlain by organic, compressible soils that may consolidate under the structure's load and cause unwanted settlement. Additional information on geotechnical data and design can be found in EM 1110-2-1903, EM 1110-2-2906, and Eckert and Callender (1987).

Existing structures

An inventory of existing structures in the project vicinity and data on their design and functional performance will assist in the design of a detached breakwater system. Depending on their proximity and influence on the study area, these structures may need to be incorporated into the design of the new project.

Review of Functional Design Procedures

Design process

Because of limited prototype experience, detached breakwater design in the United States relies on a significant amount of engineering judgement, data from a few existing breakwater projects for comparison, and an understanding of basic coastal processes. The design process is an iterative one. An initial breakwater configuration is assumed based on past experience at existing breakwater sites and taking into account the site-specific concerns and parameters described in the previous sections. This design is evaluated relative to the project's objectives, predicted beach response, and potential effects on adjacent shorelines; modifications are then made to the initial design and the project is reevaluated. Initial design should start by considering incident wave energy flux to determine the extent of wave energy reduction necessary to develop the desired beach planform.

Tools for design evaluation

Design techniques or evaluation tools for detached breakwaters can be classified into three categories: physical and numerical models, empirical methods, and prototype assessment (Rosati 1990). Numerical and physical models, when calibrated and verified at a particular project site, can effectively simulate coastal response to a particular breakwater design. Modeling, particularly numerical, is recommended prior to the implementation of the breakwater project. The use of numerical and physical modeling as tools in functional breakwater design is discussed in Chapter 3, *Tools for Prediction of Morphologic Response*. Models, however, can be more expensive and time-consuming than required for feasibility-level studies. Empirical "desktop" methods provide quick techniques for qualitatively evaluating beach response to a particular project design (Rosati 1990). The use of these simplified, inexpensive methods is desirable in the feasibility stage of project design; in the design of more extensive laboratory, numerical model and field testing; and as a check for detailed evaluation results.

Dally and Pope (1986) suggest a three-phase breakwater design process: first, a desktop study employing various empirical relationships to relate proposed structural and site parameters to shoreline response and identify design alternatives; second, a physical or numerical model study to assess and refine alternatives; and finally, if feasible, a prototype test to verify and adjust the preliminary design.

Prototype breakwater database

A database of detached breakwater projects in the United States and several other countries is maintained by CERC. The database contains information such as type of breakwater, dates of construction, project dimensions, and other site data. A brief narrative description of the project's performance is also included. Because limited design guidance exists, experience from prototype sites such as those contained in the database may prove valuable for the design of a new breakwater project.

Review of Empirical Methods

A desktop study using empirical relationships is recommended as the first step in the design of a detached breakwater system. Empirical relationships are somewhat limited due to their inherent simplicity; however, they can be used as reasonable methods prior to detailed studies to quickly assess prototype response and/or project costs for several design alternatives and as a means of assessing model results.

Numerous laboratory, numerical, and prototype studies have focussed on detached breakwaters with the objective of developing and improving functional design guidance. As a result, a number of empirical relationships for the design and prediction of beach response to single or segmented detached breakwater systems have been developed. Most investigations present information on when tombolos will form and when minimal beach response can be expected. Table 3 presents a summary of studies whose empirical methods have been used to design both U.S. and foreign detached breakwater projects. Detailed information on various empirical relationships is presented in Rosati (1990) and summarized in CETN III-43 (Coastal Engineering Research Center 1984) and EM 1110-2-1617. It is recommended that Rosati (1990) and/or the original reference be reviewed prior to using any of these empirical methods for prototype design.

EM 1110-2-1617 presents conditions for the three types of beach response as predicted by the various relationships described in Table 3. This information is summarized here, and is presented in terms of a dimensionless breakwater length L_s/y, where L_s is the breakwater segment length, and y is the distance from the average shoreline. Tables 4, 5, and 6 present conditions for tombolo development, salient development, and limited response, respectively.

Evaluation of empirical methods. Rosati (1990) conducted an evaluation of empirical design methods that consisted of compiling data from five U.S. breakwater projects, and comparing the prototype response with empirical predictions where possible. These projects encompass a range of structural and site parameters and beach response, from salient formation (Lakeview Park, Lorain, Ohio, and Redington Shores, Florida), to no sinuosity (Lakeshore Park, Ohio), to periodic tombolo formation (Colonial Beach, Central and Castlewood Park Sections, Virginia).

The majority of relationships are of the type that predict a limited, salient, or tombolo response as a function of structural parameters. Rosati (1990) conducted an evaluation of these relationships as presented in Figure 24.

In general, the simplicity of the empirical methods evaluated and prototype data limitations resulted in widely varying predictions for most design relationships. However, several of the evaluated relationships proved to predict prototype response, although careful consideration must be given to

Table 3
Empirical Relationships for Detached Breakwater Design

Inman and Frautschy (1966)	Predicts accretion condition; based on beach response at Venice in Santa Monica, CA
Toyoshima (1972, 1974)	Recommends design guidance based on prototype performance of 86 breakwater systems along the Japanese coast
Noble (1978)	Predicts coastal impact of structures in terms of offshore distance and length; based on California prototype breakwaters
Walker, Clark, and Pope (1980)	Discusses method used to design the Lakeview Park, Lorain, OH segmented system for salient formation; develops the Diffraction Energy Method based on diffraction coefficient isolines for representative waves from predominant directions
Gourlay (1981)	Predicts beach response; based on physical model and field observations
Nir (1982)	Predicts accretion condition; based on performance of 12 Israeli breakwaters
Rosen and Vadja (1982)	Graphically presents relationships to predict equilibrium salient and tombolo size; based on physical model/prototype data
Hallermeier (1983)	Develops relationships for depth limit of sediment transport and prevention of tombolo formation; based on field/laboratory data
Noda (1984)	Evaluates physical parameters controlling development of tombolos/salients; especially due to on-offshore transport; based on laboratory experiments
Shore Protection Manual (1984)	Presents limits of tombolo formation from structure length and distance offshore; based on the pattern of diffracting wave crests in the lee of a breakwater
Dally and Pope (1986)	Recommends limits of structure-distance ratio based on type of shoreline advance desired and length of beach to be protected
Harris and Herbich (1986)	Presents relationship for average quantity of sand deposited in lee and gap areas; based on laboratory tests
Japanese Ministry of Construction (1986); also Rosati and Truitt (1990)	Develops step-by-step iterative procedure, providing specific guidelines towards final design; tends to result in tombolo formation; based on Japanese breakwaters
Pope and Dean (1986)	Presents bounds of beach response based on prototype performance; response given as a function of segment length-to-gap ratio and effective distance offshore-to-depth at structure ratio; provides beach response index classification
Seiji, Uda, and Tanaka (1987)	Predicts gap erosion; based on performance of 1,500 Japanese breakwaters
Sonu and Warwar (1987)	Presents relationship for tombolo growth at the Santa Monica, CA breakwater
Suh and Dalrymple (1987)	Gives relationship for salient length given structure length and surf zone location; based on lab tests and prototype data
Berenguer and Enriquez (1988)	Presents various relationships for pocket beaches including gap erosion and maximum stable surface area (i.e., beach fill); based on projects along the Spanish coast
Ahrens and Cox (1990)	Uses Pope and Dean (1986) to develop a relationship for expected morphological response as function of segment-to-gap ratio

Table 4
Conditions for the Formation of Tombolos

Condition	Comments	Reference
$L_s/y > 2.0$		SPM (1984)
$L_s/y > 2.0$	Double tombolo	Gourlay (1981)
$L_s/y > 0.67$ to 1.0	Tombolo (shallow water)	Gourlay (1981)
$L_s/y > 2.5$	Periodic tombolo	Ahrens and Cox (1990)
$L_s/y > 1.5$ to 2.0	Tombolo	Dally and Pope (1986)
$L_s/y > 1.5$	Tombolo (multiple breakwaters)	Dally and Pope (1986)
$L_s/y > 1.0$	Tombolo (single breakwater)	Suh and Dalrymple (1987)
$L_s/y > 2\ b/L_s$	Tombolo (multiple breakwaters)	Suh and Dalrymple (1987)

Table 5
Conditions for the Formation of Salients

Condition	Comments	Reference
$L_s/y < 1.0$	No tombolo	SPM (1984)
$L_s/y < 0.4$ to 0.5	Salient	Gourlay (1981)
$L_s/y = 0.5$ to 0.67	Salient	Dally and Pope (1986)
$L_s/y < 1.0$	No tombolo (single breakwater)	Suh and Dalrymple (1987)
$L_s/y < 2\ b/L_s$	No tombolo (multiple breakwaters)	Suh and Dalrymple (1987)
$L_s/y < 1.5$	Well-developed salient	Ahrens and Cox (1990)
$L_s/y < 0.8$ to 1.5	Subdued salient	Ahrens and Cox (1990)

Table 6
Conditions for Minimal Shoreline Response

Condition	Comments	Reference
$L_s/y \leq 0.17$ to 0.33	No response	Inman and Frautschy (1966)
$L_s/y \leq 0.27$	No sinuosity	Ahrens and Cox (1990)
$L_s/y \leq 0.5$	No deposition	Nir (1982)
$L_s/y \leq 0.125$	Uniform protection	Dally and Pope (1986)
$L_s/y \leq 0.17$	Minimal impact	Noble (1978)

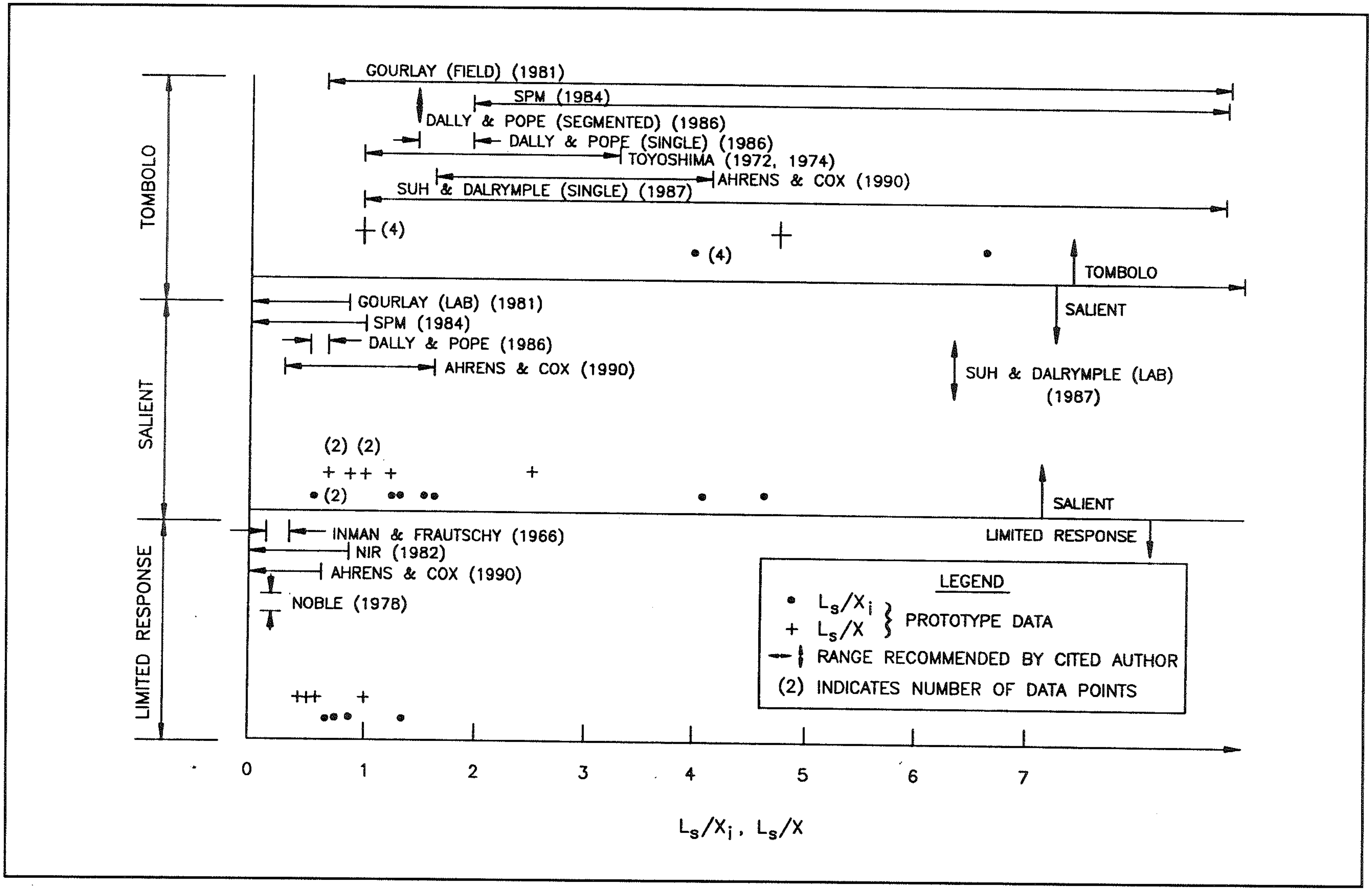


Figure 24. Evaluation of morphological relationships (modified from Rosati (1990))

their reliability and limitations throughout the design process. A number of the relationships that were evaluated are presented herein. Rosati (1990) presents additional evaluations and provides correlation coefficients for the various comparisons. Parameter definitions are provided in Appendix A.

Prediction of shoreline response. The most investigated effect of detached breakwaters is the relationship between project accretion, in particular morphological response, and structural parameters. An evaluation of these relationships showed an apparent trend in the prototype data for deposition to increase as the structure length-to-distance offshore ratio increases (Rosati 1990).

Suh and Dalrymple (1987) developed the following relationship for the prediction of salient length X_s by combining movable-bed laboratory results with prototype data:

$$X_s = X(14.8)\frac{L_g X}{L_s^2}\, e^{\left(-2.83\sqrt{\frac{L_g X}{L_s^2}}\right)} \tag{3}$$

where X is defined as the breakwater segment distance from the original shoreline and L_g is the gap distance between adjacent breakwater segments.

Tombolos usually formed for single prototype breakwaters when

$$\frac{L_s}{X} \geq 1.0 \tag{4}$$

For multiple offshore breakwaters, tombolos formed when

$$\frac{L_g X}{L_s^2} \approx 0.5 \tag{5}$$

For evaluation, Equation 3 was applied to all segmented projects. The relationship tends to overpredict the seaward excursion of the spit for themajority of prototype data evaluated, but appears to accurately predict response for pocket-beach type structures with periodic tombolo formations (Figure 25).

Prediction of gap erosion. Seiji, Uda, and Tanaka (1987) give the following gap erosion relationships, where gap erosion is defined as the retreat of shoreline to the lee of the gap from the initial (pre-project) shoreline position:

$$\frac{L_g}{X} < 0.8 \quad \text{no erosion opposite gap} \tag{6}$$

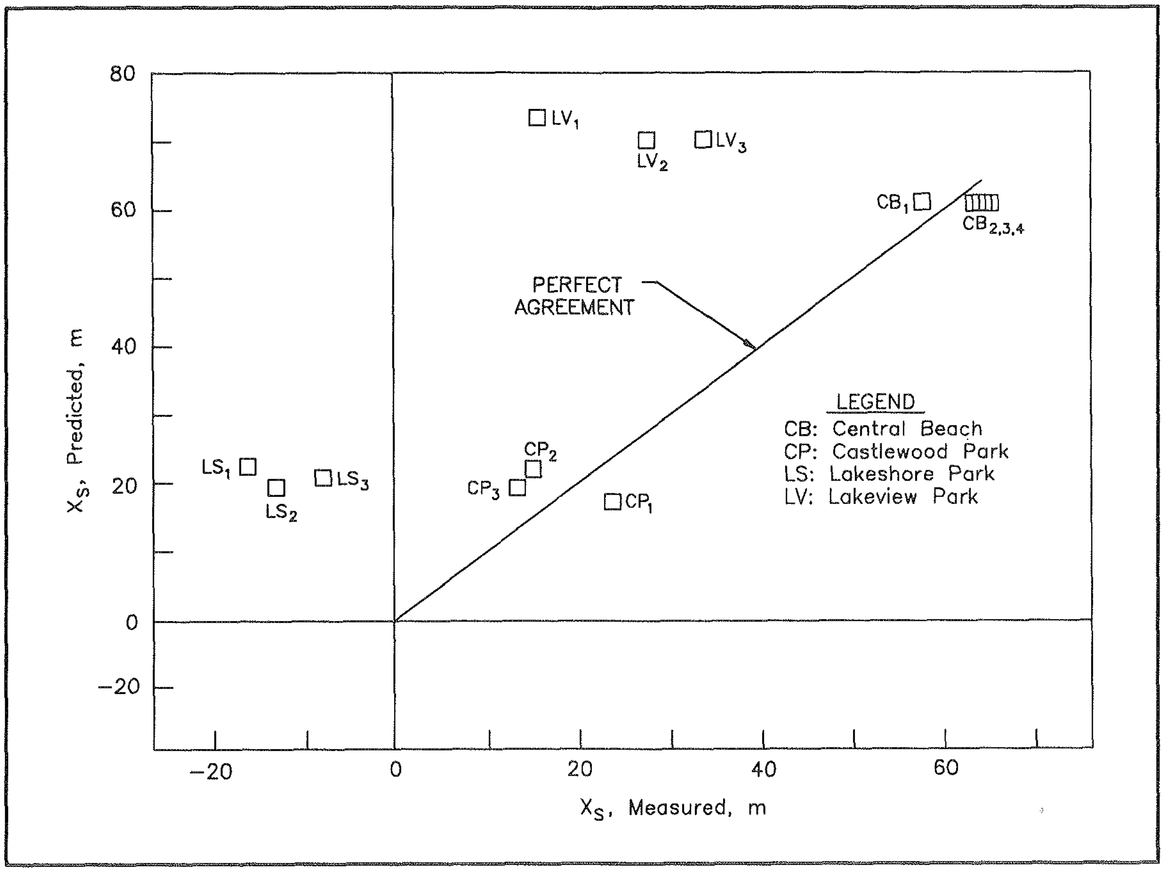


Figure 25. Evaluation of Suh and Dalrymple's (1987) relationship for salient length (from Rosati (1990))

$$0.8 \leq \frac{L_g}{X} \leq 1.3 \quad \text{possible erosion opposite gap} \tag{7}$$

$$\frac{L_g}{X} \geq 1.3 \quad \text{certain erosion opposite gap} \tag{8}$$

These relationships were evaluated with prototype data (Figure 26). The lower boundary for no erosion ($L_g/X < 0.8$) was a good predictor of either accretion or very little erosion. Gap erosion occurred for ratios of L_g/X greater than 0.8.

Structure depth. Hallermeier (1983) recommends the following water depth as a guide for positioning detached breakwaters when tombolo formation is deemed undesirable:

$$d_{sa} = \frac{2.9 H_e}{\sqrt{(S - 1)}} - \frac{110 H_e^2}{(S - 1)\, g\, T_e^2} \quad \text{depth for salient formation} \tag{9}$$

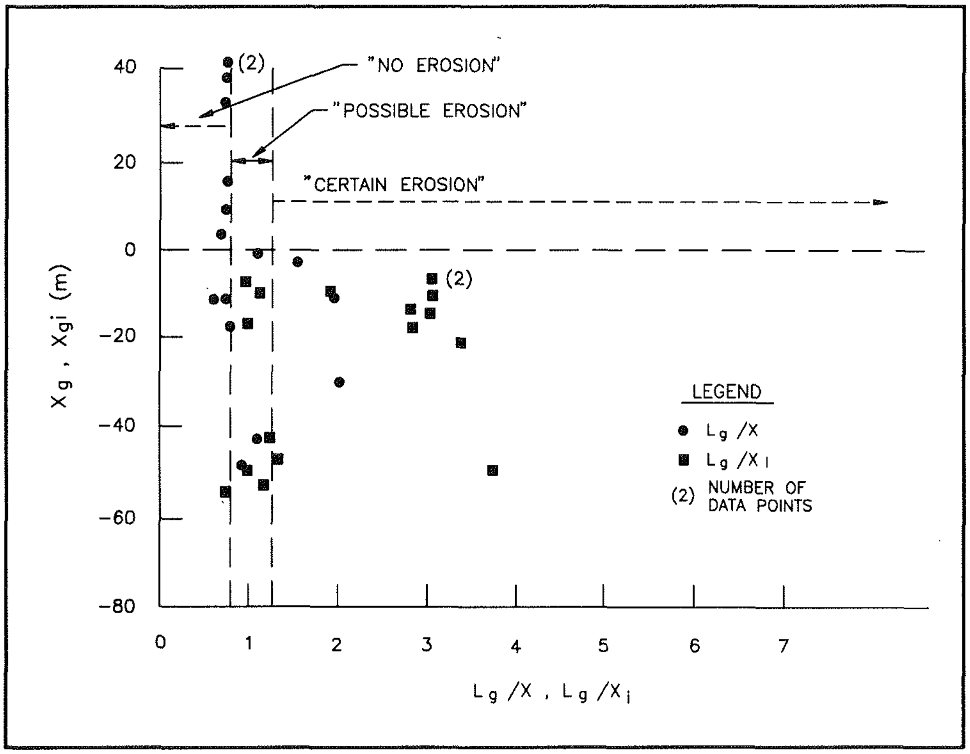


Figure 26. Evaluation of Seiji, Uda, and Tanaka's (1987) limits for gap erosion (from Rosati (1990))

where d_{sa} is the annual seaward limit of the littoral zone, H_e is the deepwater wave height exceeded 12 hr per year, S is the ratio of sediment to fluid density, g is the acceleration of gravity, and T_e is the wave period corresponding to the wave height.

For headland structures (tombolo formation), structures should be sited at an approximate depth of

$$d = \frac{d_{sa}}{3} \quad \text{headland structures} \tag{10}$$

This relationship was evaluated using the recommended depth for salient formation at all sites except Colonial Beach, where the recommended depth for tombolo formation was used. A good correlation between depth at the structure and Hallermeier's recommended depth exists for all but the Lakeshore Park data (Figure 27).

Japanese Ministry of Construction (JMC) method. The JMC method is a step-by-step iterative procedure with specific guidelines to follow during the breakwater design process (Japanese Ministry of Construction 1986; Rosati 1990; Rosati and Truitt 1990; EM 1110-2-1617). The procedure used for design is advantageous over the limited design guidance available in the United States; however, the method has several disadvantages for design of

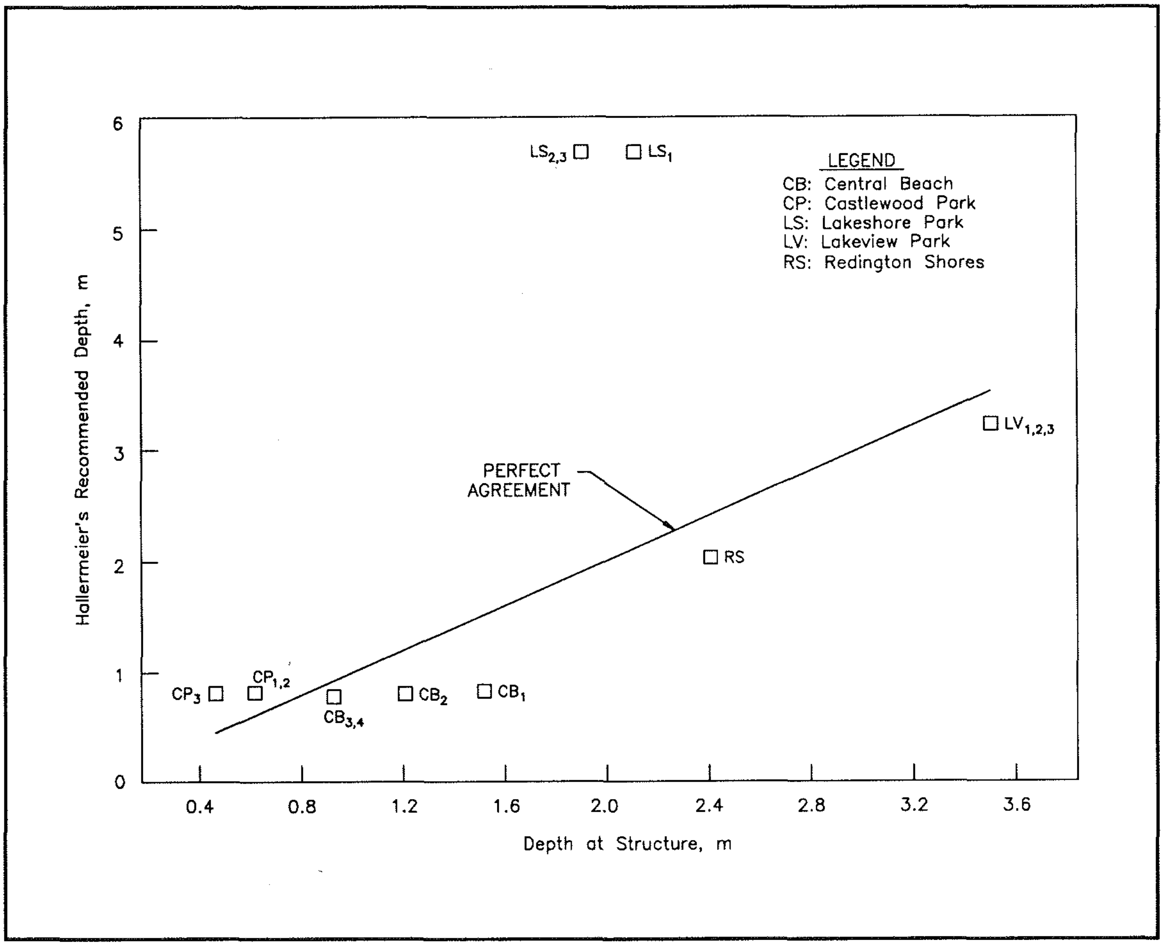


Figure 27. Evaluation of Hallermeier's (1983) relationship for structure design depth (from Rosati (1990))

U.S. projects. First, 60 percent of the projects designed using the JMC method result in tombolos (Rosati 1990), generally undesirable for projects, except for headland or pocket beach design. Secondly, the JMC method does not account for beach fill in the design, nor does it allow the designer to vary structural transmissibility.

A comparison of the JMC method and the design from the Lakeview Park project was conducted by Rosati and Truitt (1990). For the four example problems and site parameters evaluated, use of the JMC design tended to result in more numerous, shorter length segments with a decreased gap width. Additionally, the JMC structures are designed to be placed closer to shore than the distance observed in U.S. projects.

Evaluation of methods using Lakeview Park. The Lakeview Park project was used to intercompare relationships and further assess their validity (Rosati 1990). The Diffraction Energy Method (Walker, Clark, and Pope 1980) was used to design this project, which has been successful in terms of shoreline protection. A comparison of as-constructed project parameters to those recommended by the JMC method and Toyoshima's median-depth system (Toyoshima 1972, 1974) was conducted. Both of these methods resulted in segment lengths and gap distances smaller than the constructed project, with structures positioned closer to shore than indicated by the Diffraction Energy Method.

Empirical methods used in U.S. design

This section briefly describes four methods presented in Table 3 that have commonly been used in the design of more recent U.S. breakwater projects. These methods were not specifically evaluated with prototype data by Rosati (1990), and were therefore excluded from the previous section. Two of the methods, Pope and Dean (1986) and Ahrens and Cox (1990), are applied in the case example presented in Appendix B.

Dally and Pope (1986). Dally and Pope present several techniques for controlling shoreline response to a single or segmented detached breakwater project. They recommend the following limits for the structure length-distance offshore ratio (and gap distance for segmented systems) based on the type of beach planform desired and the length of beach to be protected.

For tombolo development:

$$\frac{L_s}{X} = 1.5 \text{ to } 2 \quad \text{single breakwater} \tag{11}$$

$$\frac{L_s}{X} = 1.5,\ L \leq L_g \leq L_s \quad \text{segmented breakwater} \tag{12}$$

where L is the wavelength at the structure.

For salient formation:

$$\frac{L_s}{X} = 0.5 \text{ to } 0.67 \quad \text{single and segmented breakwaters} \tag{13}$$

For uniform protection over a long distance and an unconnected shoreline, a structure located outside of the surf zone is recommended. Either a permeable (60 percent), partially submerged structure or an impermeable, frequently segmented structure will allow ample wave energy into the area. In order to provide sufficient distance for the diffracted waves to reorient themselves via refraction before reaching the shoreline, the recommended ratio for a segmented system is:

$$\frac{L_s}{X} < 0.125 \quad \text{segmented breakwaters} \tag{14}$$

Lengthening the structure or reducing its distance offshore beyond the condition given in Equations 13 and 14 will increase the extent of the tombolo and assure tombolo development. This, however, may eventually form a double tombolo planform with trapped water between, which may lead to undesirable water stagnation problems. To further assure tombolo development, the breakwater should be constructed to prevent or minimize wave transmission through the structure. Crest elevation and slope should be designed to minimize wave overtopping. Likewise, to prevent tombolo formation and allow only salients to develop, wave energy in the lee should be increased by increasing wave transmission and overtopping of the structure. Increasing gap width will also increase wave energy behind the structure.

Pope and Dean (1986). Based on prototype data, Pope and Dean (1986) defined a shoreline classification scheme that included five types of beach response: permanent tombolos, periodic tombolos, well-developed salients, subdued salients, and no sinuosity. A relationship was developed that gives the beach response classification scheme as a function of the ratios of segment length to gap length L_s/L_g and effective distance offshore to the average water depth at the structure $\bar{X}/\bar{d}_s$. Figure 28 shows the relationships between all prototype projects relative to these two dimensionless parameters. The projects plotted in Figure 28 show a grouping that may define fields of predictable beach planform response for low to moderate wave climates. It should be noted that these results are only preliminary and further verification is required.

Ahrens and Cox (1990). Ahrens and Cox (1990) used the beach response index classification scheme of Pope and Dean (1986) to develop a predictive relationship for beach response based on a ratio of the breakwater segment length to breakwater segment distance from the original shoreline. The relationship defining a beach response index I_s is:

$$I_s = e^{\left(1.72 - 0.41\frac{L_s}{X}\right)} \tag{15}$$

For the five types of beach response defined in Pope and Dean (1986), the following values of I_s were specified:

$I_s = 1$ (Permanent tombolo formation)

$I_s = 2$ (Periodic tombolos)

$I_s = 3$ (Well-developed salients)

$I_s = 4$ (Subdued salient)

$I_s = 5$ (No sinuosity)

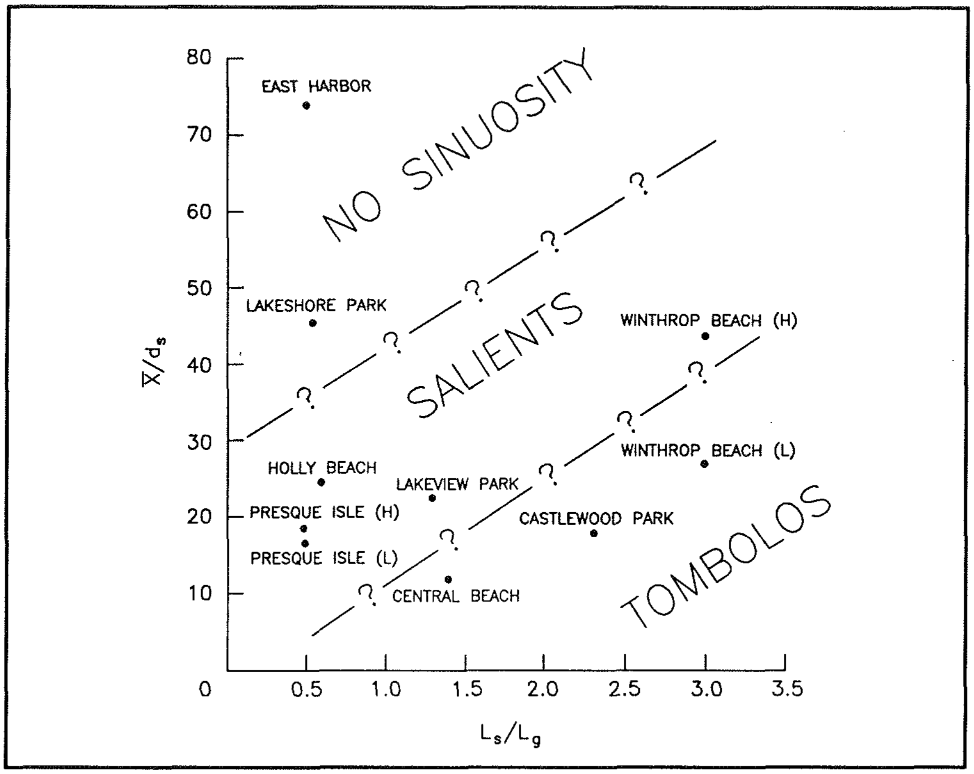


Figure 28. Dimensionless plot of United States segmented breakwater projects relative to configuration (from Pope and Dean (1986))

A breakwater design can be evaluated using this method by computing the beach response index for various combinations of breakwater lengths and offshore distances. This method is applied in the case example provided in Appendix B.

Silvester, Tsuchiya, and Shibano (1980). This method has recently been used in the functional design of headlands at Sims Park, Euclid, Ohio, by the USAED, Buffalo (1986). The spacing and location of the headland breakwaters is interrelated as shown on Figure 29, where a is the maximum indentation, b is the headland spacing, R_1 and R_2 are radii of the spiral, θ is the angle between radii R_2 and R_1 (where $R_2 > R_1$), and α is the constant angle between either radius and its tangent to the curve. The ratio of a/b is fixed for a given obliquity of incident waves to the headland alignments, β. Through successive iterations using β, the spacing and location of the breakwaters can be obtained. Further information is provided in Silvester, Tsuchiya, and Shibano (1980) and Silvester and Hsu (1993).

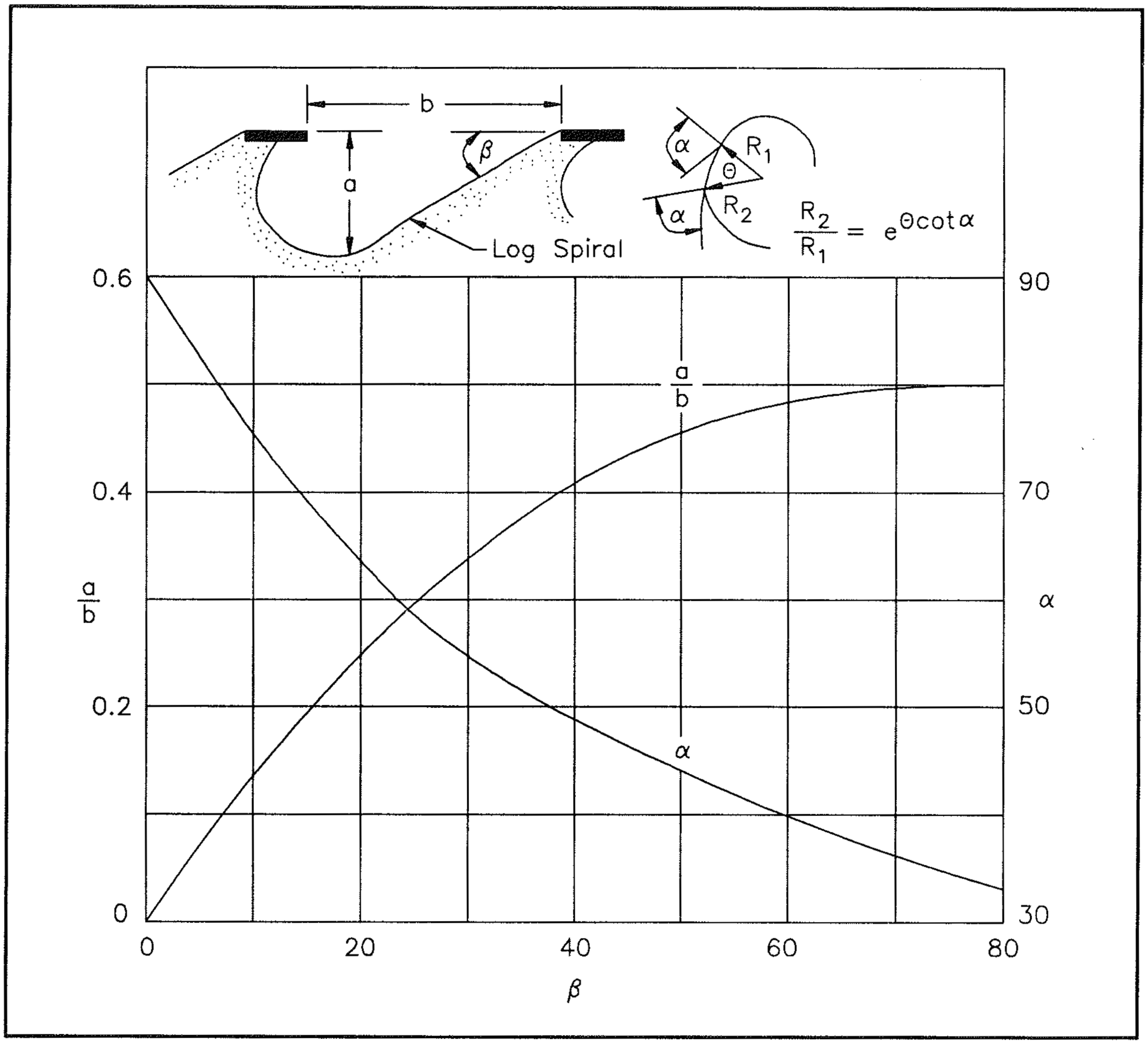


Figure 29. Parameters relating to bays in static equilibrium (Silvester, Tsuchiya, and Shibano 1980)

3 Tools for Prediction of Morphologic Response

Introduction

The knowledge base and engineering experience essential for coastal project design is developed from information on project evolution, response of other projects with similar coastal processes, and empirical relationships. From this framework, the design engineer refines the project goals and limitations, and identifies types of solutions that may be feasible at the site. Numerical and/or physical model simulation is recommended for further assessment of these design alternatives. Use of numerical and physical models facilitates unbiased evaluation and optimization of alternatives, as well as providing a structure that assists in directing data collection and analysis. The purpose of this chapter is to discuss numerical and physical modeling as applied to detached breakwater design, provide general guidelines for conducting these studies, and present examples of model use with prototype detached breakwater projects.

Numerical Models

Overview

Beach change numerical models use sediment transport relationships and conservation of volume to simulate beach response to various driving forces (e.g., waves, currents, and water levels). There are two types of well-tested beach change models: short-term (hours to days) storm-induced profile prediction, and long-term (months to decades) shoreline response models (Kraus 1990).

Correct application of a storm-induced beach profile change model requires the assumption that longshore transport is constant for the project reach, and all beach change occurs in the cross-shore direction. These models are primarily employed to design and evaluate beach fill projects, in conjunction with the shoreline change models. Shoreline response models assume that longshore sediment transport is the primary long-term contributor to planform response. The underlying postulation is that cross-shore movement of sediment during storms equilibrates over the long term. Shoreline response models are best applied to sites for which there is a clear trend in beach change. Thus, shoreline change models are well-suited to predict morphologic response of the beach as a function of detached breakwater design. However, for those detached breakwater projects with beach fill that are intended to provide storm protection, storm-induced profile change models may also be applied in the design process to provide a worst-case evaluation of beach fill response to extreme events. For more information on the Storm-Induced BEAch CHange (SBEACH) model available from CERC, see CERC (1993), Larson and Kraus (1989), and Larson, Kraus, and Byrnes (1990).

One-line or shoreline response models idealize the beach profile with an average shape, which moves seaward or landward as the beach accretes or erodes, respectively. The shoreline response model available from CERC, GENESIS (Hanson and Kraus 1989b; Gravens, Kraus, and Hanson 1991) will be discussed herein because of its wide availability and previous application to detached breakwater projects. GENESIS may be obtained as an executable file for personal computer (PC) use (Gravens 1992), or applied within the Coastal Modeling System (CMS) documented by Cialone et al. (1992).

By making simplifying assumptions to one-line modeling theory, analytical or closed-form solutions to the mathematical models may be derived. Larson, Hanson, and Kraus (1987) present more than 25 closed-form solutions for predicting beach evolution and structure interaction. Included are solutions for salient evolution behind a detached breakwater, and the final equilibrium shoreline position. These solutions can provide a simple and economical means of making a rapid qualitative evaluation of shoreline response.

Another class of numerical model that has assisted in detached breakwater design is the multi-contour line model. This type of model can describe the evolution of a number of beach contours to varying waves and currents, both in the longshore and cross-shore directions. These models have not yet been widely applied; they require considerable modeling expertise and computational capability. Of note was an application of the "N-Line Model" (Perlin and Dean 1983, Scheffner and Rosati 1987, Scheffner 1988) to provide qualitative results for use in functional design of the Redington Shores, Florida, detached breakwater project (USAED, Jacksonville 1984). Three-dimensional models are at the forefront of beach change simulation research, and will eventually allow the most detailed description of nearshore evolution. These models calculate sediment transport rates as a function of waves, currents, and resulting changes in bathymetry at many points defined by a horizontal grid. Because of their complexity, these models require detailed input and calibration data sets, powerful computers for application, and extensive verification and sensitivity testing (Kraus 1990).

GENESIS

Assumptions. Shoreline change models generally have five basic assumptions: (a) a constant beach profile shape, (b) constant shoreward and seaward limits of the profile, (c) sediment transport is described as a function of breaking waves, (d) the detailed structure of nearshore circulation is neglected, and (e) a long-term trend in shoreline evolution (Hanson and Kraus 1989b). For wave transformation calculations, GENESIS assumes that the beach profile conforms to an equilibrium profile shape,

$$D = Ay^{2/3} \tag{16}$$

in which D is the water depth and A is an empirical scale parameter that relates to the median beach grain size as follows:

$$\begin{aligned} A &= 0.41 D_{50}^{0.94} \quad for \quad D_{50} < 0.4mm \\ A &= 0.23 D_{50}^{0.32} \quad for \quad 0.4mm \le D_{50} < 10.0mm \\ A &= 0.23 D_{50}^{0.28} \quad for \quad 10.0mm \le D_{50} < 40.0mm \\ A &= 0.46 D_{50}^{0.11} \quad for \quad 40.0mm \le D_{50} \end{aligned} \tag{17}$$

Consequently, only one point on the profile is required to determine its shape; this point is typically taken as the mean high water shoreline.

Sediment is assumed to be transported alongshore between two well-defined elevations on the profile, the top of the active berm at the shoreward limit, and the depth of closure offshore. Longshore sediment transport in GENESIS is determined with an empirical formula,

$$Q = H_b^2 C_{gb} \left(a_1 \sin 2\theta_{bs} - a_2 \cos\theta_{bs} \frac{\partial H_b}{\partial x} \right) \tag{18}$$

in which H_b is the breaking wave height; C_{gb} is the wave group speed at breaking, given by linear wave theory; θ_{bs} is the angle of breaking waves to the local shoreline; and x is the longshore coordinate. The non-dimensional coefficients a_1 and a_2 are given by

$$a_1 = \frac{K_1}{16(S-1)(1-p)} \tag{19a}$$

$$a_2 = \frac{K_2}{8(S-1)(1-p)\tan\beta} \tag{19b}$$

where K_1 and K_2 are empirical coefficients, S is the ratio of the density of sand to the density of water, p is the porosity of sand on the bed, and $tan\beta$ is the average bottom slope from the shoreline to the depth of active longshore sand transport. The coefficients K_1 and K_2 are treated as model calibration parameters, with K_2 on the order of 0.5 to 1.0 times K_1. Both K_1 and K_2 control the magnitude and rate of shoreline change in the model, although the importance of K_2 is apparent in the vicinity of coastal structures, where diffraction produces a substantial change in breaking wave height over a short longshore distance (Gravens, Kraus, and Hanson 1991).

Capabilities. The capabilities and limitations of GENESIS Version 2.0 are detailed by Gravens, Kraus, and Hanson (1991). GENESIS can simulate shoreline change due to an almost arbitrary number of engineering works, alone or in combination: detached breakwaters, groins (T-shaped, Y-shaped, and spur), jetties, seawalls, and beach fills. The model simulates sand bypassing around groins and jetties, and has the capability to simulate diffraction and wave transmission at groins, jetties, and detached breakwaters. Offshore waves may be input with arbitrary height, period, and direction, and may be described as multiple wave trains (as from independent sources, e.g., sea and swell). Sand transport is predicted both due to oblique wave incidence and longshore gradients in wave height. The model may be applied to a project with wide spatial extent (from hundreds of meters to tens of kilometers).

Limitations. General shoreline change modeling assumptions as presented previously limit GENESIS applicability to situations for which these assumptions are reasonable representations of the project site and planned use. In addition, GENESIS does not simulate wave reflection from structures. The shoreline can not touch a detached breakwater; therefore, tombolo evolution at detached breakwaters or a headland breakwater system can not be modeled. There are minor restrictions on placement, shape, and orientation of the structures, and the model does not directly provide for changing tide level. GENESIS is not applicable to calculating shoreline change for situations in which beach change occurs unrelated to Equation 18, such as: in the vicinity of inlets or areas dominated by tidal currents; regions for which wind-driven beach transport is significant; storm-induced beach change for which cross-shore transport processes are dominant; and scour at structures (Hanson and Kraus 1989b).

Data requirements

Two levels of physical data are typically required prior to conducting shoreline change modeling; background information used to make an assessment of coastal processes at the site on the local and regional levels, and project-level information with which the model can be calibrated, verified, and applied to examine future scenarios. The first level includes information about regional transport rates, regional geology, water levels (typical ranges and datums), and the frequency and extent of extreme events. Analysis of these data allows the modeling simulations to be evaluated in larger spatial and temporal contexts.

The project-level information includes shoreline position data (representing at least three different times for model calibration and verification), offshore waves, beach profiles and offshore bathymetry, and information on structures and other engineering activities (both past and planned works). Beach profiles are required to determine the average shape of the beach, and offshore bathymetry is used to transform the offshore wave data to nearshore values. The shoreline position data are required for calibration, verification, and application of the model. Calibration requires that shoreline position data be available for two different times, together with waves corresponding to that time period. The model parameters K_1 and K_2, and in some cases with detached breakwaters, the structure transmission to incoming wave energy, K_T, are determined to reproduce known shoreline change. Model verification refers to using the second shoreline position with the calibrated parameters to predict a third shoreline. Once again, waves representative of conditions that occurred to cause evolution of the shoreline from the second to third positions should be used for model simulation. If model verification does not adequately represent the known shoreline change, the modeler must iterate through the calibration/ verification process until a reasonable model agreement with measured shoreline position is obtained. The specifications and date of engineering activities are required to properly set up the model and, for planned work, evaluate future scenarios.

Detailed discussions on the development of input data sets for use with GENESIS are given by Hanson and Kraus (1989b) and Gravens (1991). Gravens (1991, 1992) presents application of the Shoreline Modeling System, which consists of a set of analysis programs that may be used separately or in conjunction with GENESIS to streamline data preparation and analysis prior to model implementation. Specific issues relating to input data required for modeling of morphologic response to detached breakwaters are presented in a subsequent section of this chapter.

Previous GENESIS detached breakwater applications

Sensitivity testing. Hanson and Kraus (1990) investigated the effects of varying site and structure design parameters on beach response for a single detached breakwater. Simulation results lend a general understanding to how several of the controlling design variables affect beach response. The discussion presented herein is summarized from Hanson and Kraus (1990).

The structure used in the first set of GENESIS simulations was an impermeable 300-m-long breakwater, placed 300 m offshore in the 3-m water depth. The first case examined the effect of increasing offshore significant wave height from 0.2 m to 1.0 m for normally incident wave crests, while holding the wave period constant at 4 sec, for a 100-hr simulation (Figure 30). As offshore wave height increases, the transport potential of the

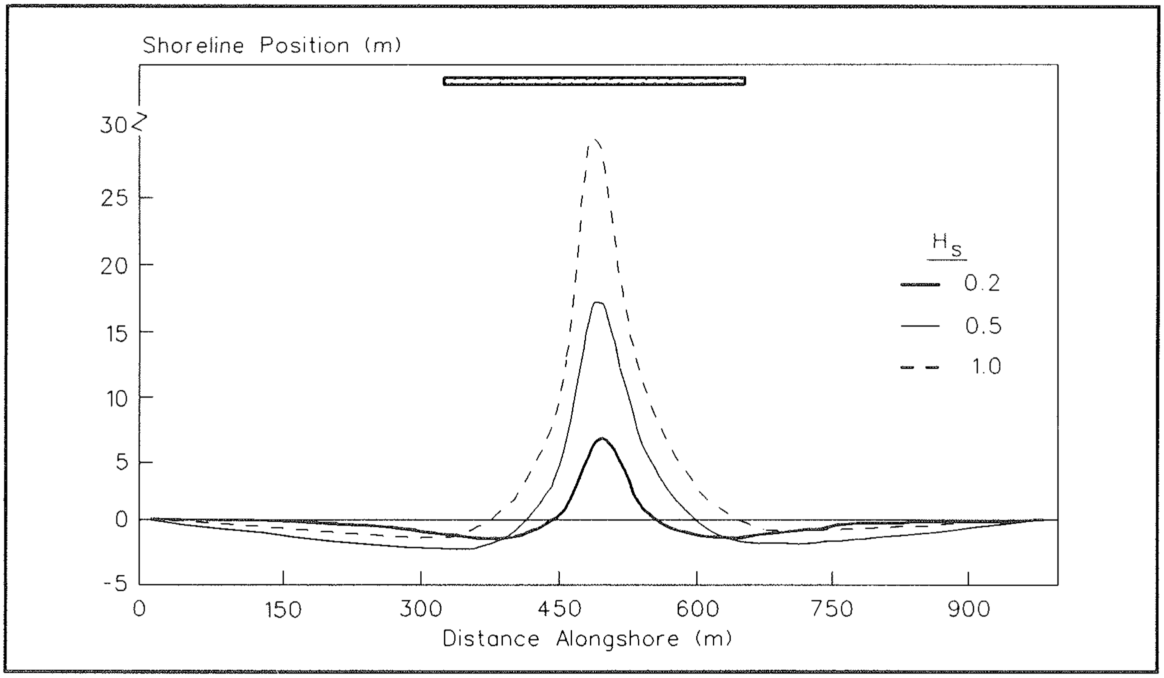


Figure 30. Influence of varying wave height on shoreline change behind a detached breakwater (Hanson and Kraus 1990)

diffracted waves in the lee increases, thus prograding the salient towards the structure. The salient progresses approximately linearly with the increase in wave height, although the accumulated volume for the larger wave height is approximately an order of magnitude larger than for the smallest wave height.

Using the same structure as discussed above, the second test examined the effects of increasing wave period on beach response. A 1-m offshore significant wave height was used with a wave period varying from 3 to 5 sec. The salient growth is shown to increase with increasing wave period (Figure 31). The longer period waves have a greater shoaling coefficient, which causes them to break further offshore, in turn resulting in a greater breaker height. As discussed above, increasing the breaking wave height advances the salient towards the structure.

The third test series examined the effects of wave variability on morphologic response (Figure 32). A 200-m-long breakwater located 200 m offshore in the 2-m depth was used for the simulations. A 1-m wave height, 4-sec wave period, approaching the initial shoreline normally (0 deg) was used for one of the wave climates; the other three simulations held two of these parameters constant while the third was normally distributed as a percentage of its mean value (see Figure 32). Results indicate that allowing the wave period and wave height to vary has little effect on the observed shoreline response. Variation of these parameters simply redistributes the wave energy in time, without changing the total longshore wave energy flux. However, increasing variability in the wave direction greatly progrades the

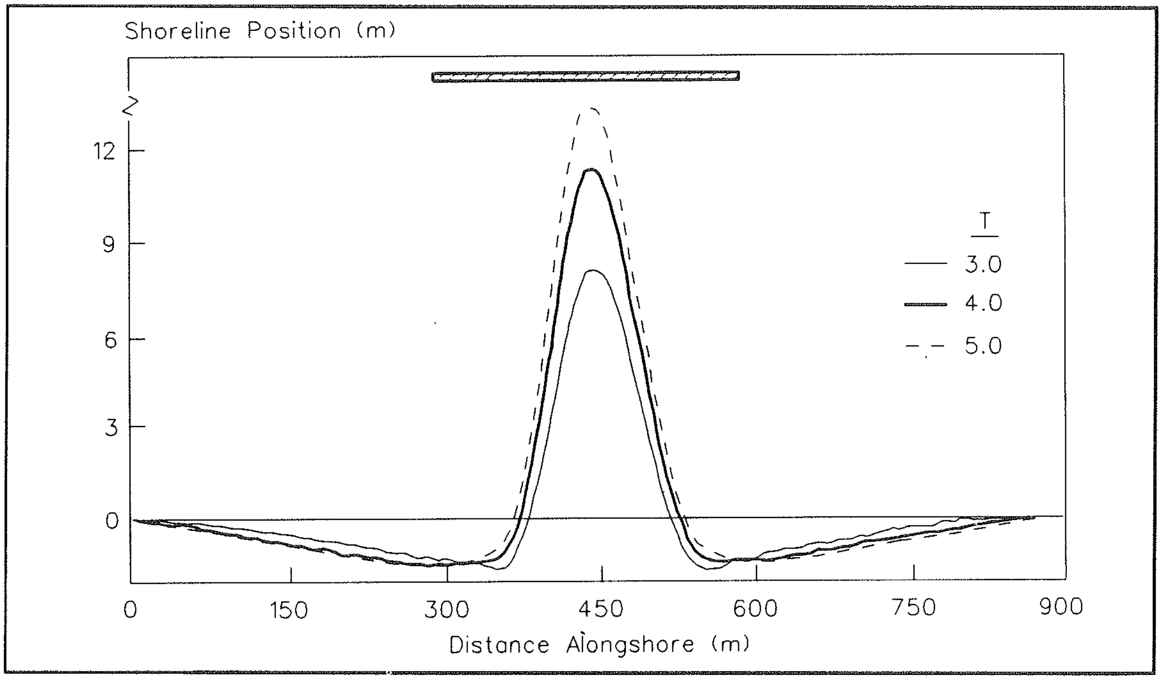


Figure 31. **Influence of varying wave period on shoreline change behind a detached breakwater (Hanson and Kraus 1990)**

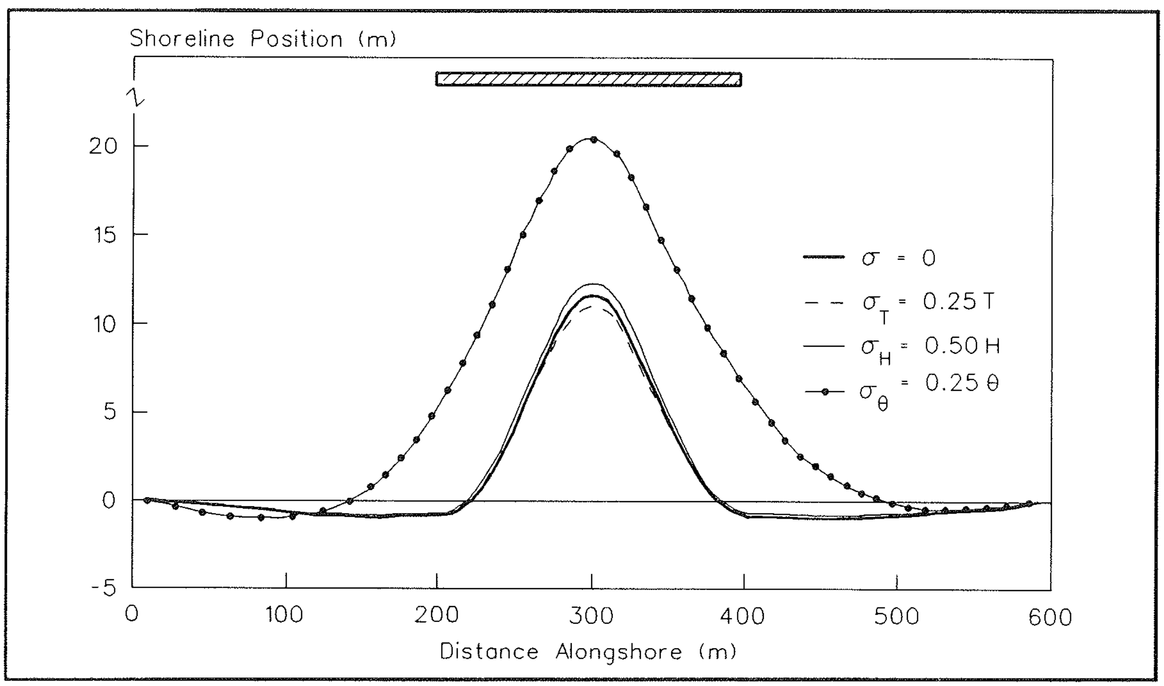


Figure 32. **Influence of wave variability on shoreline change behind a detached breakwater (Hanson and Kraus 1990)**

predicted salient. Small deviations in wave direction increase the longshore component of wave energy flux, which results in more sand being moved alongshore. Because of the sheltering provided by the impermeable structure, this transported material tends to collect in the protected region to the lee of the structure, advancing the salient seaward.

The final sensitivity test used a 200-m-long breakwater located 250 m offshore to evaluate the effect of varying structure transmission on predicted beach response. Normally incident waves with a 1.5-m significant wave height and 6-sec wave period were used in the 180-hr simulation. In GENESIS, a structure transmission K_T value of 0 indicates an impermeable structure, whereas a value of 1 indicates a structure that is transparent to incoming waves. This sensitivity test used four K_T values ranging from 0 to 0.8 (Figure 33). As expected, an impermeable structure ($K_T = 0$) results in greater salient growth, while the more permeable tests show less salient progradation. For example, $K_T = 0.2$ decreases the maximum shoreline advance 36 percent from the impermeable structure simulation, and reduces the accumulated volume by 25 percent.

Hanson, Kraus, and Nakashima (1989) also present example calculations illustrating GENESIS's breakwater transmission capability. A three-segment system, each segment with a different transmission coefficient, is used to simulate beach response as a function of varying wave approach. A second test series uses a continuous structure with varying transmission properties alongshore, which might occur in nature due to differential settling of the structure, or uneven loss of armor stone.

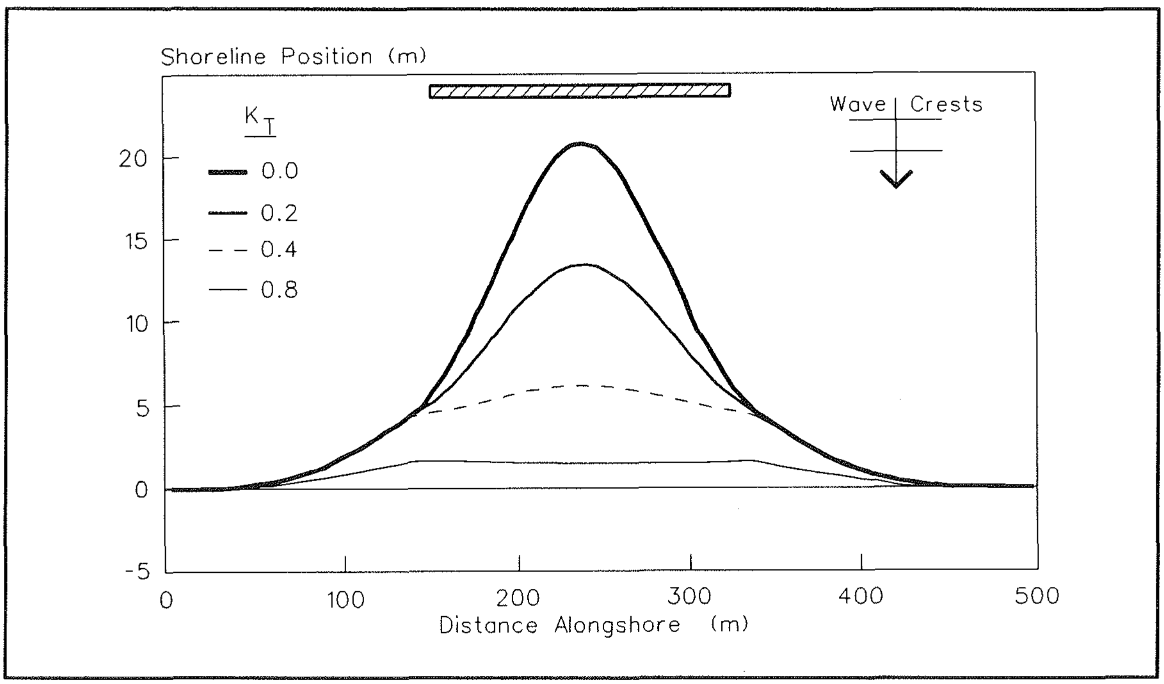


Figure 33. Shoreline change as a function of transmission (Hanson, Kraus, and Nakashima 1989)

Hanson and Kraus (1990) extended these sensitivity tests by applying GENESIS in a generalized manner for the case of a single detached breakwater. They developed a nomograph predicting morphologic response as a function of several dimensionless design parameters (deepwater wave height over depth at structure, breakwater length over wave length at the structure, and structure transmission), which compared favorably with six prototype detached breakwater projects. Rosati, Gravens, and Chasten (1992) continued this work to develop nomographs for single and segmented detached breakwaters. However, since these studies were conducted, a limitation within GENESIS was identified which indicated that the nomographs may tend to overpredict tombolo formation[1]. The nomographs presented by Hanson and Kraus (1990) and Rosati, Gravens, and Chasten (1992) may be useful in indicating dependencies on controlling dimensionless parameters. However, they are not recommended for application to project design in their present form.

Site-specific examples. Application of GENESIS to two detached breakwater projects is summarized from existing literature. Discussion of these studies herein is directed towards providing the engineer steps involved in numerical modeling of detached breakwater systems. For details about each application, the referenced publications should be consulted. In addition, Appendix B discusses the application of GENESIS at the Bay Ridge, Maryland, detached breakwater project.

(1) Holly Beach, Louisiana. Hanson, Kraus, and Nakashima (1989) demonstrated use of the breakwater transmission capabilities of GENESIS through preliminary calibration results with the Holly Beach, Louisiana, detached breakwater project. The project consists of six detached breakwater segments, each with a different cross-sectional design. The structures are constructed of various quantities and arrangements of timber piles, used tires, and riprap, which result in varying degrees of wave transmission.

The first step in the modeling project was to gather and evaluate all relevant data sets and previous studies. Ten grid cells are recommended behind each detached breakwater, thereby requiring a cell spacing of 4.6 m. From available shoreline change data, it was determined that there were locations of minimal movement outside the project area. Therefore, the "pinned beach" boundary condition (see Hanson and Kraus (1989b); Gravens, Kraus, and Hanson (1991) for details) was applied at the ends of the project reach, to allow sand transport in and out of the calculation domain. Based on field dye studies of structure permeability, the structure transmission coefficients were qualitatively known to generally decrease from east to west. The western-most segment was riprap, and showed little wave transmission, whereas the eastern-most segment consisted of tires mounted on one row of timber piles, and had the greatest observed dye transmission (Nakashima et al.

---

[1] Personal Communication, 1992, Mr. Mark B. Gravens, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

1987). Using these results, Hanson, Kraus, and Nakashima (1989) initially set the K_T values to 0.9, 0.3, 0.5, 0.3, 0.7, and 0.1 for segments from east to west, respectively. The parameters K_1 and K_2 were set to 0.5 and 0.1, respectively, after initial testing. Wave data from an offshore gauge were used to create an 18-month time series corresponding to the available survey data. Because this time period included effects of Hurricane Bonnie, which made landfall close to the gauge, the wave data required extensive censoring to eliminate spurious extremes and give reasonable estimates of wave conditions at the site. The resulting mean wave height and period at the gauge after modification of the data set were 0.53 m and 5 sec, respectively.

During the calibration process, the K_T values were modified to 0.4, 0.8, 0.2, 0.1, 0.0, and 0.0 for segments from east to west, respectively. The authors expected to change these values slightly, since their initial estimates were based on visual observations of dye movement, whereas structure transmission pertains to wave heights and directions. The calculated shoreline position agreed well with the measured position, with locations and widths of salients well-reproduced (Figure 34). The shoreline change corresponding to the gap regions was not predicted so well. The calibrated model was used to predict beach response for a three-year time series. Qualitatively, this prediction compared reasonably well with survey data.

(2) Lakeview Park, Lorain, Ohio. Hanson and Kraus (1989a, 1991) present simulations of shoreline response to the three-segment detached breakwater project at Lakeview Park, Lorain, Ohio. The purpose of this application was to provide field verification of GENESIS for transmissive

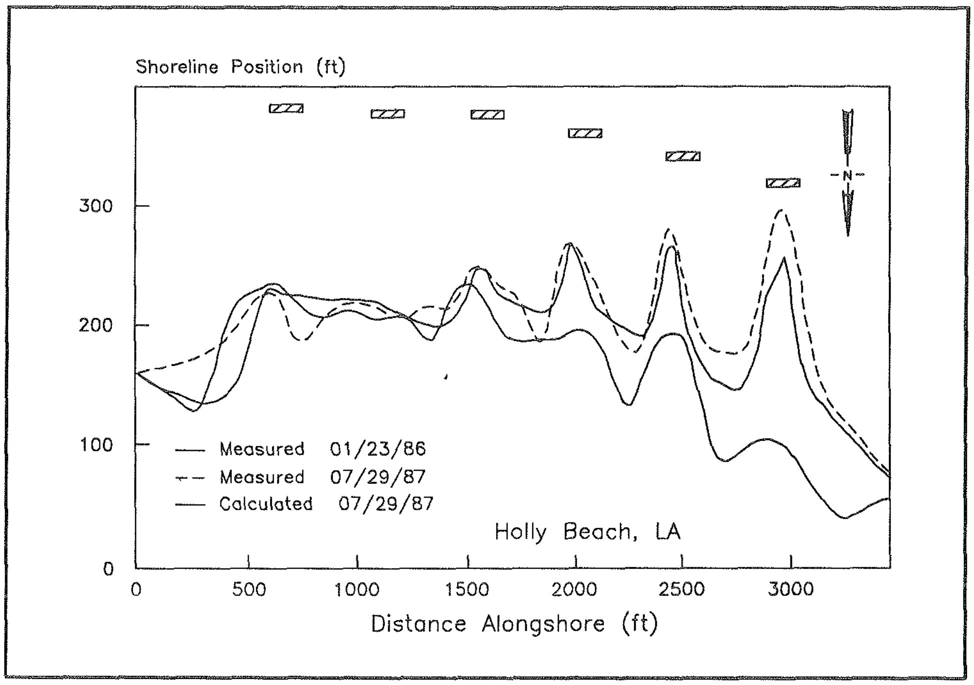


Figure 34. Preliminary model calibration, Holly Beach, Louisiana (Hanson, Kraus, and Nakashima 1989)

detached breakwaters. The Lakeview Park project consists of three rubble-mound detached breakwater segments protecting a placed beach fill, contained by groins on both ends.

Again, the first step in the numerical modeling application was to evaluate all relevant studies and data sets available for the project area. Ten calculation cells were set up behind each detached breakwater, for an alongshore grid spacing of 7.6 m. A one-year wave time series was used in the calibration. First, the parameter K_1 was initially varied until calculated overall net transport rates were close to estimated values. Then K_2 and the boundary condition at the west groin were varied to obtain the approximate magnitudes of sand inflow from the west. Then the transmission coefficients were varied to achieve correct salient sizes, with a best fit obtained for K_T = 0.50, 0.22, and 0.30 from west to east. Finally, the location of the eastern breakwater was translated two grid cells to the east to obtain better agreement of the eastern-most salient position. The result provided good agreement between calculated and measured shoreline positions (Figure 35), with a mean absolute difference of 1.2 m. Calculated and measured volumetric changes also compared well (3,360 cu m predicted versus 3,290 cu m measured).

For the model verification time period, the boundary at the west groin was observed from aerial photographs to have retreated approximately 18 m. Therefore, this boundary condition was altered in the model setup, and model verification proceeded using a 13-month wave time series. Reasonable agreement was obtained, although sensitivity testing indicated that increasing the wave heights by 10 percent would result in a more accurate prediction, as shown in Figure 36. The mean absolute difference between measured and calculated shoreline positions was approximately 1.2 m, and the calculated and measured volumetric change compared well (-238 cu m calculated, -256 cu m measured).

Prior to evaluating alternative structure configurations, model sensitivity to key parameters should be examined. The authors investigated sensitivity of the calibrated model to variations in K_1, K_2, D_{50}, and K_T. Of particular note is that, when grain size was halved, the predicted shoreline position indicated an increase in sand volume. This is due to the more gently sloping equilibrium beach profile used in GENESIS, which moves the breaker line further offshore. However, GENESIS does not account for cross-shore movement of material, which would be greater for the smaller diameter fill. Using an average value for K_T for each segment produced acceptable results, although differences in transmission coefficients could be possible due to differences in breakwater construction, wave transformation across an irregular bottom, and differential settling.

The authors used the verified model to evaluate alternative designs for maintaining the beach fill in place. Simulations with detached breakwaters only, groins only, and groins extended further seaward were evaluated. Hanson and Kraus conclude that the combination of detached breakwaters and short groins, as constructed, is superior to simpler designs for preserving

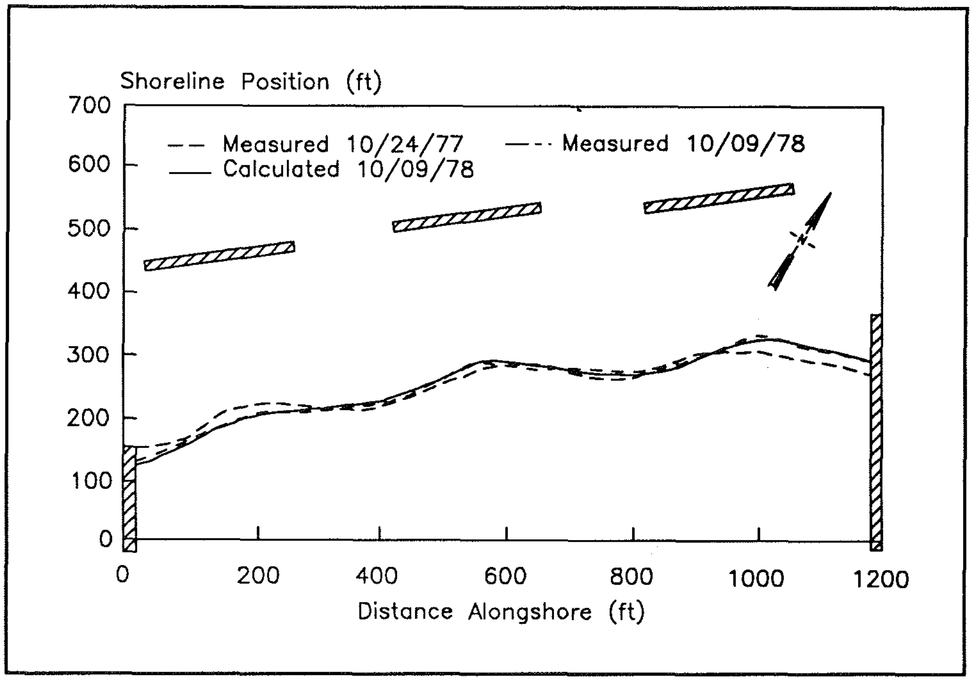


Figure 35. Calibration at Lakeview Park, Lorain, Ohio (Hanson and Kraus 1991)

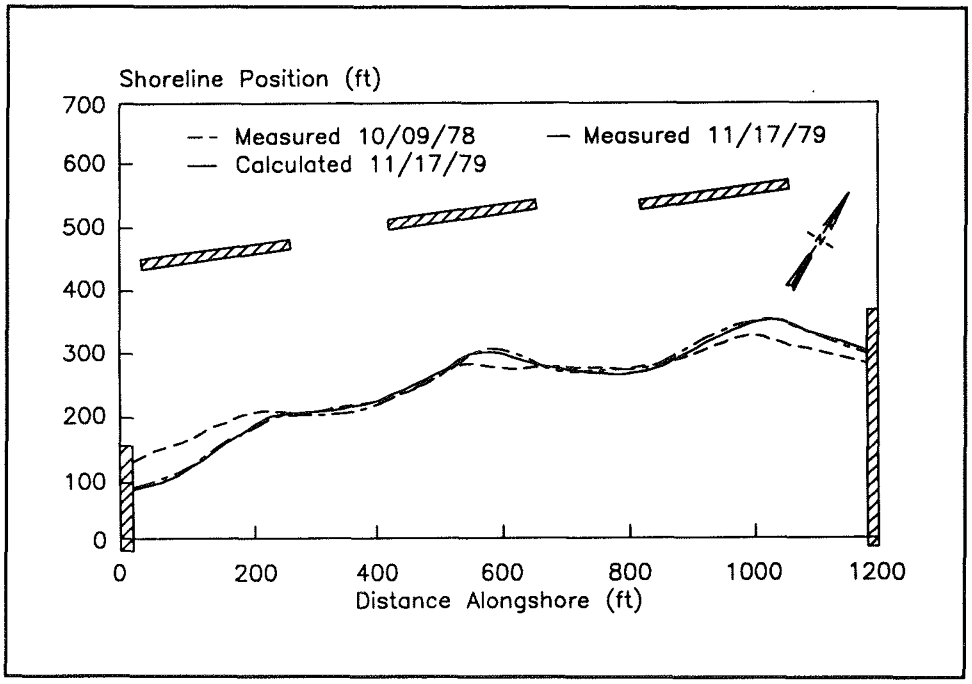


Figure 36. Verification at Lakeview Park, Lorain, Ohio (Hanson and Kraus 1991)

placed beach fill material. The model was then applied to a five-year simulation. Results were good, showing correct trends of shoreline recession on the west side, advancement on the east side, salient type, and a pivot point in the shoreline where the position did not vary over time.

Summary. Table 7 summarizes the GENESIS modeling parameters for the detached breakwater projects discussed above and presented in Appendix A. The best modeling parameters to use for shoreline change modeling studies are those that recreate known shoreline change through the calibration and verification process. However, calibration and verification data sets are not always available, and parameters must be estimated based on previous studies. Selected input data are briefly reviewed below.

Table 7
GENESIS Modeling Parameters for Detached Breakwater Studies

Project Name	Number of Breakwater Segments	Transmission Coefficients	Transport Parameter K_1	Transport Parameter K_2	Time-Step (hour)	Longshore Grid Spacing (m)	Median Grain Size (mm)
Holly Beach, Louisiana (Hanson, Kraus, and Nakashima (1989)	6	0.0, 0.0, 0.1, 0.2, 0.8, 0.4 (west to east)	0.50	0.10	0.25	4.6	0.20
Lakeview Park, Lorain, Ohio (Hanson and Kraus 1989b, 1991)	3	0.50, 0.22, 0.30 (west to east)	0.42	0.12	0.30	7.6	0.40
Presque Isle, Pennsylvania (USAED, Buffalo, unpublished)	3 (Bch 10) 3 (Bch 1)	0.25 (all) 0.50 (all)	0.40 0.10	0.12 0.12	3 3	7.6 7.6	1.0 1.0
Bay Ridge, Maryland (Appendix B)	11	0.10	0.50	0.25	1	3.8	0.5

(1) Structure transmission. In Version 2.0 of GENESIS, a constant wave transmission value is assigned to each breakwater segment. In nature, transmission for a given structure varies as a function of the incident wave characteristics and water levels. For GENESIS simulations of Lakeview Park and Holly Beach, the breakwater transmission coefficients were determined through the calibration process to reproduce known shoreline change. For Presque Isle, breakwater transmission coefficients were ultimately chosen based on physical model testing results. The availability of transmission data from the physical model tests in the Presque Isle study provided an opportunity to compare an average K_T with a transmission coefficient calculation using a standard calculation procedure. This comparison indicated

that, in the absence of calibration and verification data with which the transmission parameter can be determined, the transmission coefficient should be calculated using the largest waves within the period of record to more accurately model shoreline change. Relationships for estimating rubble-mound structure transmission are presented in Chapter 4.

(2) Cell spacing. Hanson and Kraus (1989b) and the individual simulations discussed herein recommend using from 8 to 10 longshore cells per breakwater segment to ensure proper resolution of shoreline change to the lee of the structure.

(3) Median grain size. As discussed previously, GENESIS uses median grain size to determine the steepness of the equilibrium profile shape. Ideally, a typical project profile should be used to back-calculate an effective grain size that produces a similar profile shape (see equilibrium profile template provided by Hanson and Kraus (1989b)). In the absence of bathymetric data, a representative median grain size in the surf zone should be specified. The user should check to ensure that the structure depth specified in the GENESIS input file approximates the profile depth corresponding to the desired distance offshore.

(4) Wave climate. Gravens and Scott (1993) compared different hindcast wave climates to measured wave gauge data for a site in Florida, and evaluated the data set that best reproduced longshore sand transport rates. They recommended that, if available, a two-component wave climate be used in numerical modeling of longshore sediment transport. A two-component wave train allows wave input from two wave sources, and more accurately represents what occurs in nature.

Physical Models

The final design of coastal structures such as a detached breakwater system is often evaluated using physical hydraulic models. These models can predict the breakwater's performance in the actual (prototype) coastal location and determine desired or necessary modifications to improve breakwater performance. Physical model results can also be used to validate results from the previously described numerical simulations. Physical model geometric scales for coastal applications typically range from 1:20 to 1:500, and in some cases near full-scale modeling or tracer studies are used to reproduce sediment movement observed at the actual site location.

Physical models exactly reproduce prototype conditions when geometric, kinematic and dynamic similarity are attained. However, complete similarity is seldom possible. Therefore, the more critical physical conditions (i.e., gravity waves, water currents, friction, surface tension, sediment motion, etc.)

are identified, and similarity is attained for the most dominant or severe condition. In some cases the geometric scale is distorted because modeling one dimension (i.e., horizontal length) may cause another dimension (i.e., depth, sediment diameter) to be extremely small or large, which is impractical or results in improper flow conditions. In general, physical hydraulic models are classified as undistorted fixed-bed, distorted fixed-bed, or movable-bed, which is usually distorted because the model and sediment scales are different.

Fixed-bed models basically mean the bed, or bottom, is not moving. Typically, the bed is constructed to the required depth contours using concrete mortar. An undistorted fixed-bed model has the same geometric scale for all length dimensions (i.e., length, width, depth, characteristic size, etc.), and geometrically distorted models scale one of the dimensions or characteristic size at a different geometric scale. For example, the width is scaled 1:100 and the depth is scaled 1:10.

In coastal applications the bed is usually sediment, and it can move as a result of the hydrodynamic forces exerted by the moving fluid medium. These forces are caused by coastal currents, waves, and water level changes. Complete three-dimensional movable bed models are the best approach to model applications where knowledge of sediment movement is desired such as is the case for determining the performance of a detached breakwater system. However, these models do not assure total similitude and the cost, complexity, and time required to conduct experiments result in modified movable bed models, which do not satisfy all of the primary similarity requirements. Therefore, a combination of fixed-bed modeling, tracer studies, and movable-bed modeling has been employed at WES.

The WES has numerous large and small physical model facilities for conducting fixed- and movable-bed model tests and sediment tracer studies. The USACE guidance for physical modeling of coastal phenomena is described by Hudson et al. (1979). Authorative references related to physical modeling or model similitude such as Langhaar (1951), Keulegan (1966), Yalin (1971), and Schuring (1977) are additional sources of guidance for the conduct of physical modeling in the laboratory. Hughes (1993) addresses fixed- and movable-bed modeling specifically for coastal engineering, with a chapter that is completely devoted to movable-bed modeling and incorporates the latest knowledge from the engineering and scientific communities. The open literature is another source for guidance and examples of physical modeling procedures and experiences. Frequently referenced studies of Kamphuis (1975), Noda (1971), and Le Mehaute (1970) describe guidelines and procedures for movable-bed modeling and tracer studies.

Summary of procedures for physically modeling shoreline response to detached breakwaters

Over the past two decades, physical modeling procedures have been developed and used by WES to evaluate detached segmented breakwaters.

These procedures are described by Curren and Chatham (1977, 1980), Bottin (1982), Seabergh (1983), and Dally and Pope (1986). A number of undistorted fixed-bed models, tracer studies, and/or movable-bed models have been constructed at WES and used to predict the performance of detached breakwaters in minimizing shoreline erosion. Fixed-bed models are used to investigate the interaction of existing structures (i.e., groins) with the planned breakwaters and their effect on wave-generated currents. Tracer studies are then used in the fixed-bed model to qualitatively illustrate sediment movement for existing structures and planned breakwater additions.

In movable-bed models, wave conditions are first generated in the movable-bed model to match or create an existing shoreline condition (base case). Then, the same wave conditions are generated with the model detached breakwater in place, and the shoreline effects are observed and documented. Several plans for the breakwater placement and/or characteristics are usually modeled to determine the optimum design.

Prototype data. Well-documented information regarding prototype conditions over a sufficient period of time is crucial for this type of modeling. Wave characteristics, water level, bathymetry, shoreline history, sediment characteristics, currents, and sediment budget are necessary. Curren and Chatham (1980) indicate that the essential data are littoral transport computation, sediment size distribution analysis, and the simultaneous measurement of incident wave characteristics, bottom bathymetry, and littoral and offshore-onshore sediment transport over a period of erosion and accretion. Movable-bed modeling requires the most field data and a minimum of two years of prototype data are recommended by Dally and Pope (1986). Since data requirements are project specific, it is important that the client and modeler communicate to determine whether the necessary data are available or need to be collected prior to the modeling effort.

Fixed-bed model. Froude similitude is normally used for fixed-bed models and the geometric scale is selected as large as possible. Factors considered in scale selection are depth of water required to prevent excessive bottom friction effects, model wave heights, available model area, wave generating and instrument capability, efficiency, and cost. The beach and bathymetry are constructed of concrete mortar to reproduce the bathymetry contours for a known prototype condition documented at some date and time. Existing shore protection structures are also constructed and incorporated in the model. For example, groins are usually constructed of galvanized metal or stone and placed in the model according to prototype maps and survey data. The detached breakwaters are constructed of stone and each stone is scaled so wave reflection and transmission are correctly modeled. The undistorted fixed-bed model of sufficient size correctly reproduces wave refraction, shoaling, diffraction, breaking, and nearshore circulation cells (rip, feeder, and eddy currents). The important parameters to be modeled are wave height, period and direction, water levels, and wave-generated currents alongshore and adjacent to structures. Dye injected into the water has been used to measure and document current patterns and magnitudes. Waves are produced by a wave generator, which can be reoriented to obtain directionality of the waves. Monochromatic waves have been used in previous studies of Curren and Chatham (1977, 1980), Bottin (1982), and Seabergh (1983), but capabilities currently exist for generating irregular waves for future studies. Results of fixed-bed modeling can assist in the determination of breakwater location to minimize rip current occurrence, scour around structures, offshore sediment transport, and hazards to swimmers. These results are also used to evaluate wave attenuation characteristics for various wave conditions, water levels, and breakwater lengths.

Tracer studies. Sediment tracer studies are conducted by placing lightweight sediment as a thin veneer over the fixed-bed bottom contours and observing the location of sediment accumulation and direction of transport. This technique was successfully used by Bottin and Chatham (1975). Selection of the tracer material is based on criteria of Noda (1971), which relates model to prototype ratios of sediment size, specific gravity, and horizontal and vertical model scales. Noda's method assumes a distorted scale exists in a movable-bed model. Because an undistorted model is used for fixed-bed modeling to accurately model wave refraction and diffraction, a range of tracer sediment sizes is determined by using the vertical scale ratio first and then the horizontal scale ratio to evaluate sediment size scale ratios for the same specific gravity. In the Presque Isle, Pennsylvania, study (Seabergh 1983), the prototype sediments varied from natural sands with median diameters ranging from 0.11 to 0.25 mm to coarser sands used for beach fill with a median diameter of 1.8 mm. Using the Noda method for crushed coal specific gravity of 1.35, the model sediments were required to be 2.05 to 2.69 times the prototype size. Therefore, the crushed coal particle diameter ranged from 0.22 mm (2.05 x 0.11) to 4.84 mm (2.69 x 1.8), and 0.5-mm crushed coal was used in the tracer study. Results may be used to evaluate the effects of breakwater distance offshore on longshore sediment transport and to duplicate qualitative tombolo development.

Movable-bed modeling. A movable-bed model section is constructed and inset in an area of the fixed-bed model. Wave conditions, water levels, and sediment size are adjusted to produce the documented prototype phenomena (base case) and then the same hydrodynamic conditions are used with the different improvement plans installed in the model, one at a time, to demonstrate effects on the shoreline. The sediment size for the model is determined by the same technique as described for the tracer tests, only different scaling criteria may be selected. In the Presque Isle study, a 0.9-mm crushed coal was used to model the beach fill sediment. The model sediment is continually fed along the shoreline interface where sediment is removed as a result of wave effects. These tests take considerable time to allow the sediment to redistribute itself and to show effects of the in-place detached breakwater structures on improving the stability of the shore material. The model results give only qualitative information on the sediment transport. Results may be used to evaluate bathymetry response to a detached breakwater and beach fill readjustment due to the breakwater.

Summary of physically modeled detached breakwater projects conducted at the USAE Waterways Experiment Station

Four physical model studies of detached segmented breakwater systems, namely Presque Isle, Pennsylvania, Lakeview Park, Ohio, Oceanside Beach, California, and Imperial Beach, California, were conducted in laboratory facilities at WES from 1978 to 1983. All modeling was performed in a movable-bed facility and is described in detail by Seabergh (1983), Bottin (1982), and Curren and Chatham (1977, 1980). A summary is presented by Dally and Pope (1986), which is the basis of the project summaries presented herein.

Presque Isle model study. Presque Isle Peninsula near Erie, Pennsylvania, is a recurved sand spit protecting Erie Harbor, and is also a state park with 11 recreational beaches along the approximately 11-km shoreline. Historically, the landward connection of the spit has been severed several times and beach erosion continued as the spit migrated to the east. Groin field and beachfill projects did not halt the erosion, and consequently, detached segmented breakwaters were considered as a possible stabilizing solution. A prototype segmented breakwater with three segments was constructed in 1978, and field monitoring of the shoreline response was conducted. These data were used to verify sediment movement in subsequent physical models.

A 1:50 scale, undistorted physical model as described by Seabergh (1983) was constructed to evaluate the performance of the segmented detached breakwaters at Presque Isle using Froude scaling laws. The model reproduced 2,865 m of shoreline that included part of an existing groin field and a relatively unstructured section of the shore as shown in Figure 37. This permitted study of the interaction of the proposed breakwaters with two beach sediment types, and of particular interest was the positioning of the breakwaters with respect to the existing groins. A movable-bed section was constructed in the model test basin using crushed coal based on sediment scaling procedures of Noda (1971). Tests were conducted for existing conditions (base plan) and three segmented breakwater plans. These tests included (1) measurement of wave-generated current and water circulation patterns, (2) crushed coal tracer tests, and (3) crushed coal beachfill tests.

A shoreline response similar to that observed in the prototype was experimentally duplicated. Figure 38 shows a comparison between the model and prototype shorelines after an accretionary period and then after the winter season when higher water levels and severe wave conditions reduced tombolo development. Figure 39 shows one of the proposed breakwater plans installed in the laboratory model. The results indicated that a 107-m spacing between 46-m-long segments produced satisfactory conditions within the reach covering the groin field. The optimum placement of the breakwaters was offshore of the groin ends. With the groin field removed, the segments could be placed closer to shore with reduced generation of offshore currents, but the tombolos took longer to form.

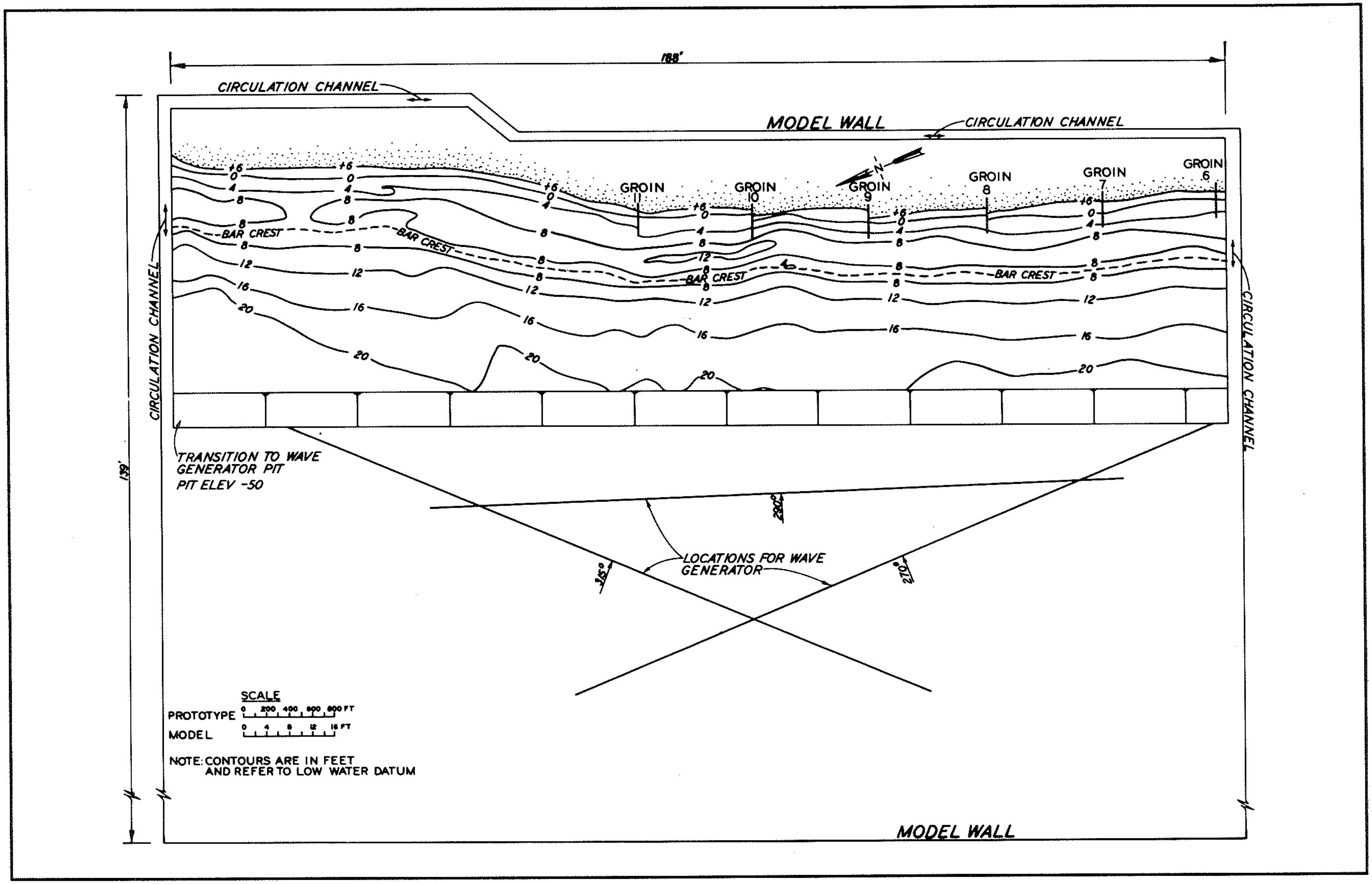
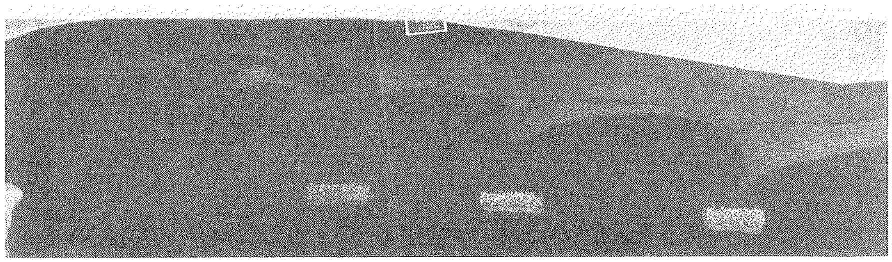
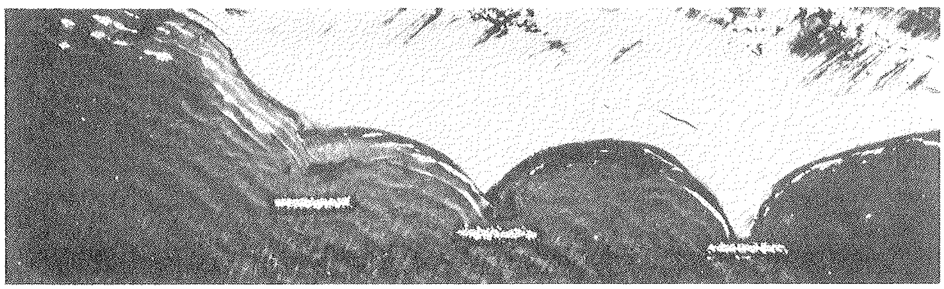


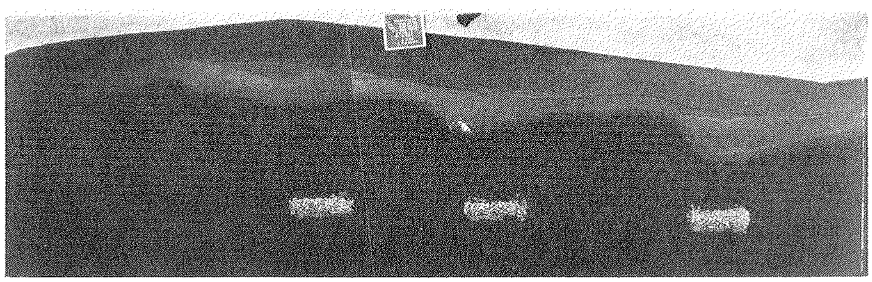
Figure 37. Layout of the Presque Isle model (multiply by 0.3048 to convert feet to meters) (Seabergh 1983)



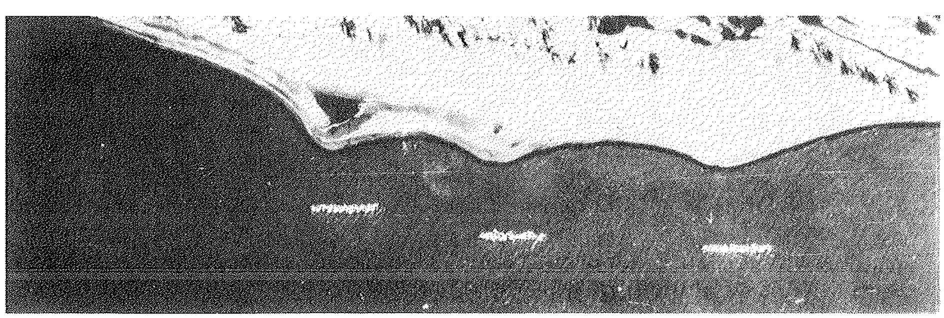
a. MODEL (ACCRETION OF SHORELINE)



b. PROTOTYPE (16 NOVEMBER 1979)



c. MODEL (EROSION OF SHORELINE)



d. PROTOTYPE (17 APRIL 1980)

Figure 38. Comparison of shoreline response for the Presque Isle model and prototype segmented detached breakwater (Seabergh 1983)

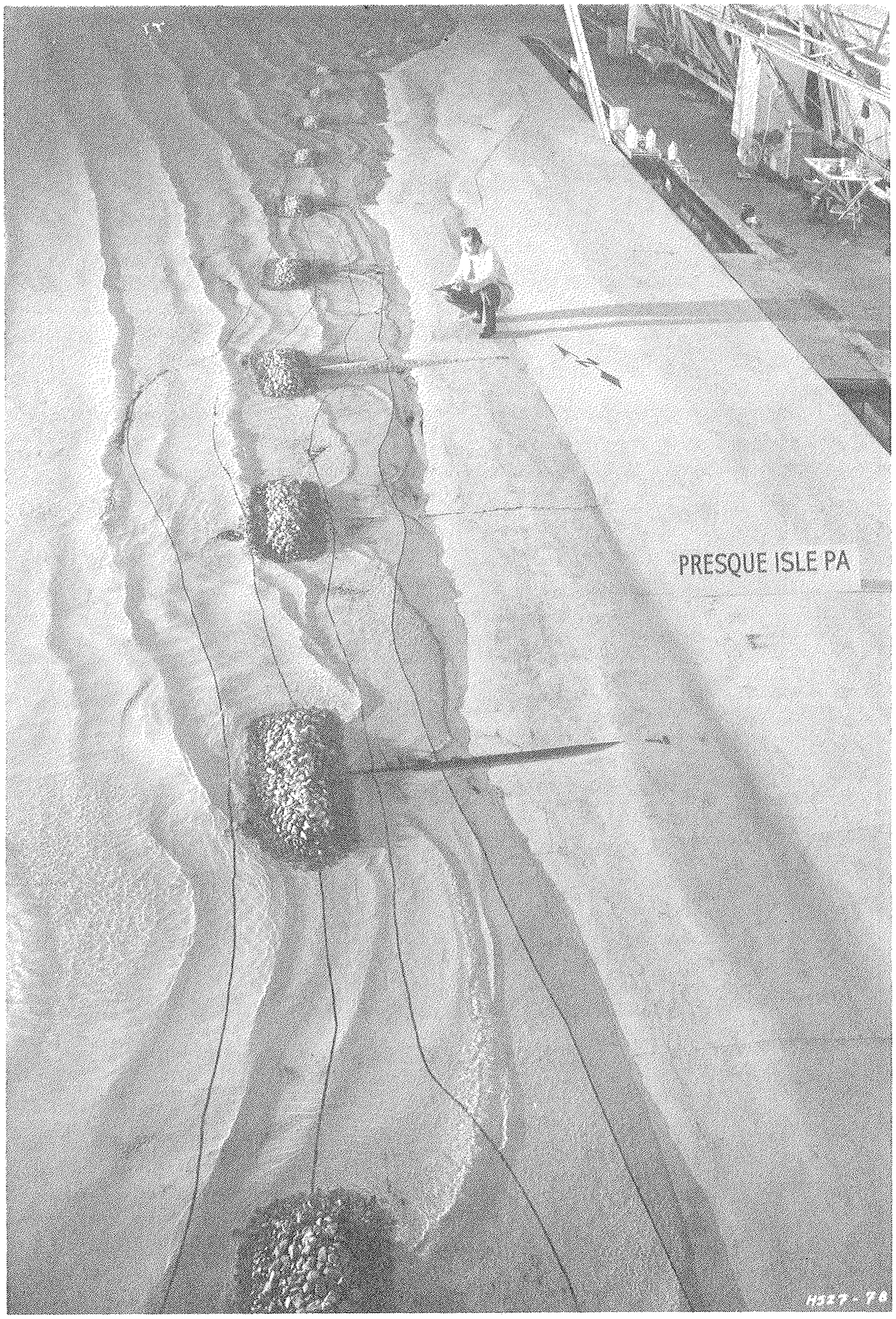


Figure 39. An example detached breakwater plan as installed in the Presque Isle model (Seabergh 1983)

Lakeview Park model study. Lakeview Park is a recreational facility located in Lorain, Ohio, along the southern shore of Lake Erie. A detached breakwater system was constructed that consisted of three 76-m-long rubble-mound segments, a 59-m-long rubble-mound extension of the east groin, an increased crest height for the landward 15-m west groin, and a placement of 84,106 m^3 of beach fill. The detached breakwaters and groin modifications were designed to protect the beach fill and shoreline. Following construction, localized erosion of the beach fill on the eastern side of the west groin occurred. The fill was replenished, but subsequently eroded to form a stable beach that was narrower than desired as shown in the aerial photograph, Figure 40.

Fixed- and movable-bed physical models and tracer studies were conducted as reported by Bottin (1982) to evaluate the degree of erosion for various Lakeview Park improvement plans. Because of limited funds, testing of the improvements was conducted using a portion of the previously described Presque Isle 1:50 scale model. The Lakeview Park structures and immediate underwater contours were installed on a section of the Presque Isle model. A portion of the fixed-bed model was replaced with crushed coal to create a movable bed depicting the Lakeview Park bathymetry contours, and still-water levels were adjusted so that depths were comparable to Lakeview Park.

Model tests were initially conducted for the as-constructed Lakeview Park shoreline. Combinations of wave height, period, direction, and still-water levels were studied to determine test conditions that produced a stabilized shoreline similar to that observed in the prototype. Next, model tests were conducted for several combinations of rubble-mound extensions of the west groin and west breakwater. The results produced a recommendation for a

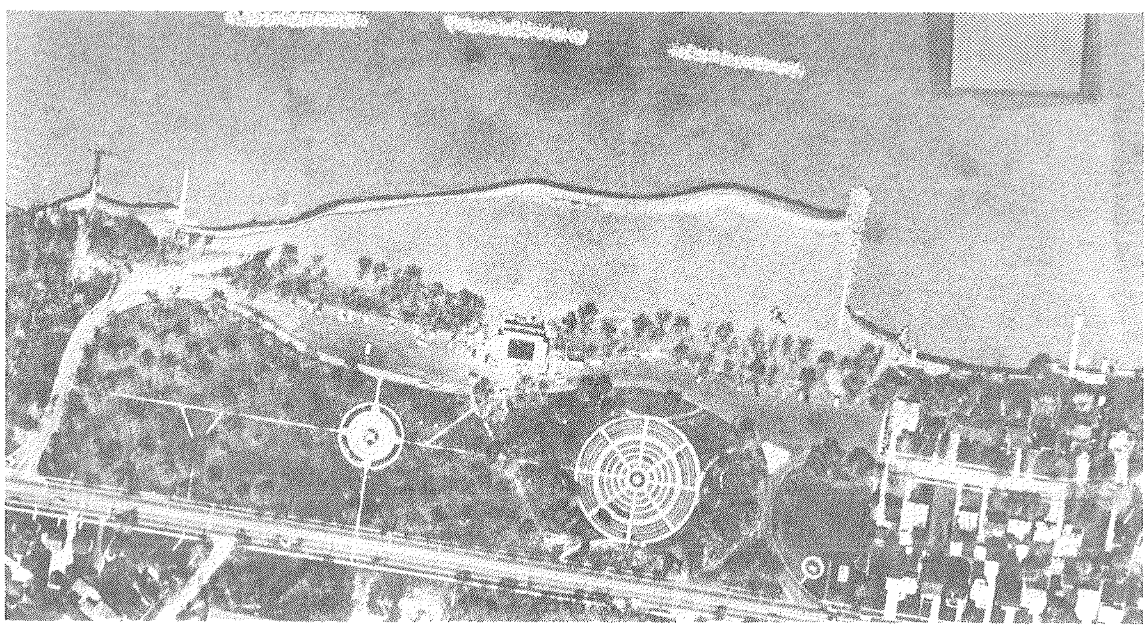


Figure 40. Aerial view of Lakeview Park in Lorain, Ohio, showing typical condition of the beach fill east of the west groin (Bottin 1982)

30.5-m-long extension of the west groin toward the western head of the west breakwater segment (Figure 41). This resulted in a smaller opening between the groin and breakwater; thus, less wave energy penetrated the opening and resulted in only a minor retreat of the west-end shoreline. The model tests are only qualitative, but showed the significance of the groin for local erosion.

Oceanside Beach model study. Oceanside Beach is a recreational beach located along the Pacific Ocean approximately 129 km southeast of Los Angeles and 48 km northwest of San Diego, California. Persistent erosion of Oceanside Beach and accretion of sand in the Oceanside Harbor and entrance channel have occurred since construction of the Del Mar Boat Basin in 1943.

As described by Curren and Chatham (1980), an undistorted fixed-bed physical model with a geometric scale of 1:100 was constructed to investigate the arrangement and design of proposed structures for preventing erosion of Oceanside Beach. Froude modeling laws and a crushed coal tracer material were used in modeling existing conditions and several improvement plans. First, tracer material was placed on the fixed-bed model surface at selected locations and fed into the longshore current to determine the mechanisms of littoral movement. Second, the tracer material was placed in a layer representing beach fill on the model surface to determine areas of accretion and erosion. However, the extent of erosion was limited by the fixed model surface. Finally, the fixed-bed contours were removed and remolded using crushed coal to obtain a movable-bed model. This type of modeling is the most reliable for determining areas of accretion and erosion, and it was used for each beach protection plan.

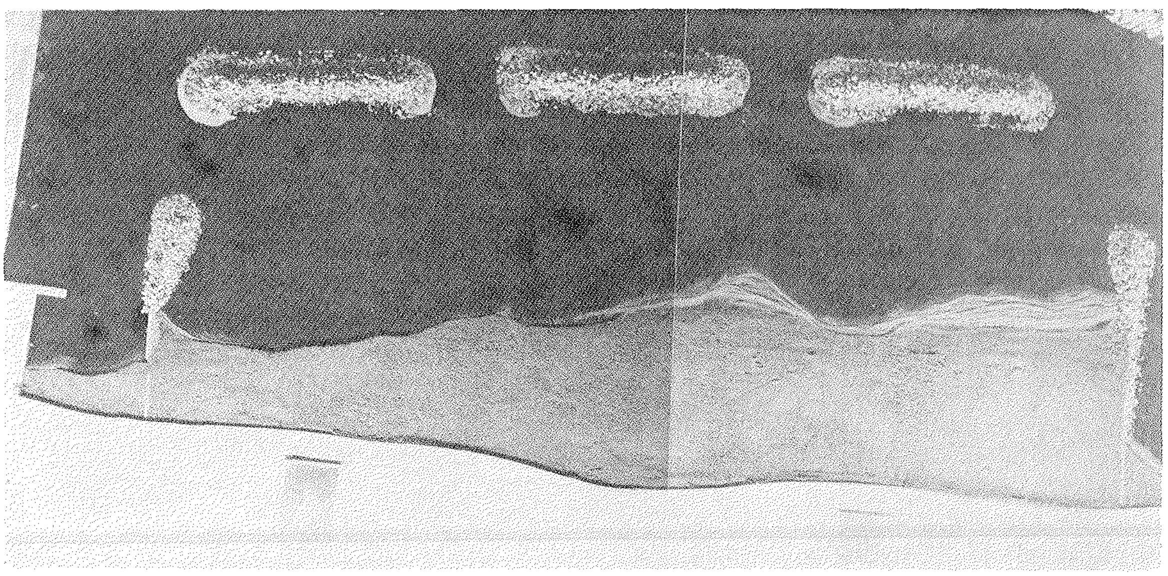


Figure 41. Shoreline in model tests with the Lakeview Park recommended plan of a 30.5-m extension of the west groin (Bottin 1982)

Modeling results for the existing condition produced an onshore movement of the coal tracer for small waves of low steepness with longshore transport at the shoreline. For high-steepness waves, the coal tracer moved seaward forming a bar at the most seaward breaker zone. This material migrated north or south depending on wave direction. The high-steepness waves re-formed and broke a second time near the shoreline, resulting in a second nearshore zone of longshore transport. The detached breakwater plans tested included a single 1,494-m-long structure with varying crest elevation and a segmented breakwater system consisting of four 203-m-long segments with 203-m gaps. Each plan was tested both with and without groins at the northern and southern extremes of the beach. Movable-bed model tests showed that the test plans without groins (Figure 42) generally resulted in erosion of the shore on the updrift side of the model beach and loss of material from the downdrift side indicating inadequate protection of the beach fill. Tests with groins at each extreme (Figure 43) showed a reduction of the amount of coal leaving the beach area and a fairly stable shore.

Imperial Beach model study. Imperial Beach is located on the Pacific Ocean coastline 5.6 km north of the Mexican border and 17.7 km south of San Diego, California. It is a recreational beach with a 366-m-long fishing pier located in the center and normal to the beach. Two groins, 226 and 122 m long, are located 899 and 495 m north of the fishing pier, respectively. The main sediment source for Imperial Beach is the Tijuana River, and construction of Morena, Barret, and Rodriquez Dams has trapped the river sediments behind the dams. Lack of river flooding has also been cited for the shortage of sediment reaching the mouth of the Tijuana River. The decreased amount of sediment available for longshore transport to Imperial Beach has caused increased beach erosion. Two groins that were constructed between 1959 and 1963 were ineffective in stabilizing the beach.

Froude model testing in a 1:75 scale physical model was conducted to evaluate the arrangement and design of alternative structures for the prevention of Imperial Beach erosion. Crushed coal was used as a tracer for modeling the existing condition and proposed new structures under various wave conditions. The proposed new structures consisted of (1) a single detached breakwater at the -4.6- and -3.0-m depth contours, (2) segmented breakwaters at the -4.6- and -1.5-m contours, (3) a single detached breakwater segmented by low sill sections at the -3.0- and -1.5-m contours, and (4) various groin locations.

The model results for existing conditions showed that both north- and south-directed longshore currents were interrupted at regular intervals by strong rip currents that transported significant quantities of sediment offshore where it was either lost in deep water, transported alongshore on a bar, or transported shoreward by low steepness waves. These model rip currents were similar to observed prototype currents. A five-groin plan resulted in strong rip currents for almost all wave conditions and was ineffective in trapping tracer material. A nine-groin series was effective in trapping tracer material, but significant quantities of stone would be required for construction.

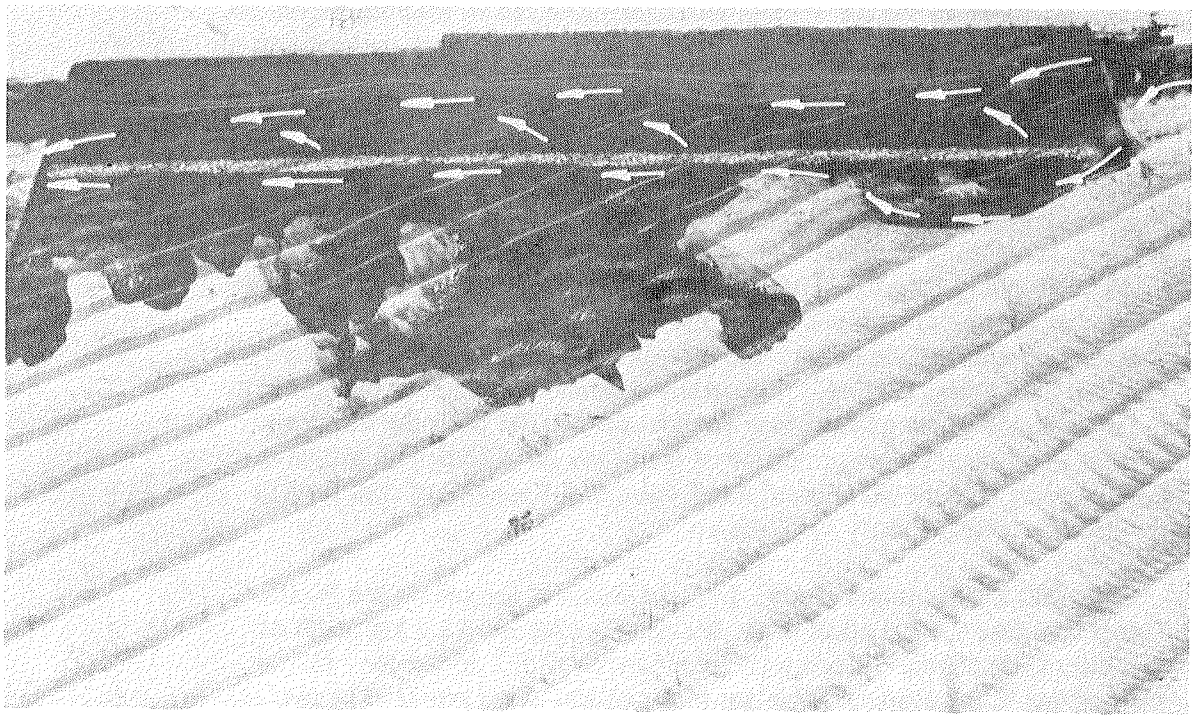


Figure 42. Oceanside Beach model test results for a single detached breakwater without groins. Arrows show current direction (Curren and Chatham 1980)

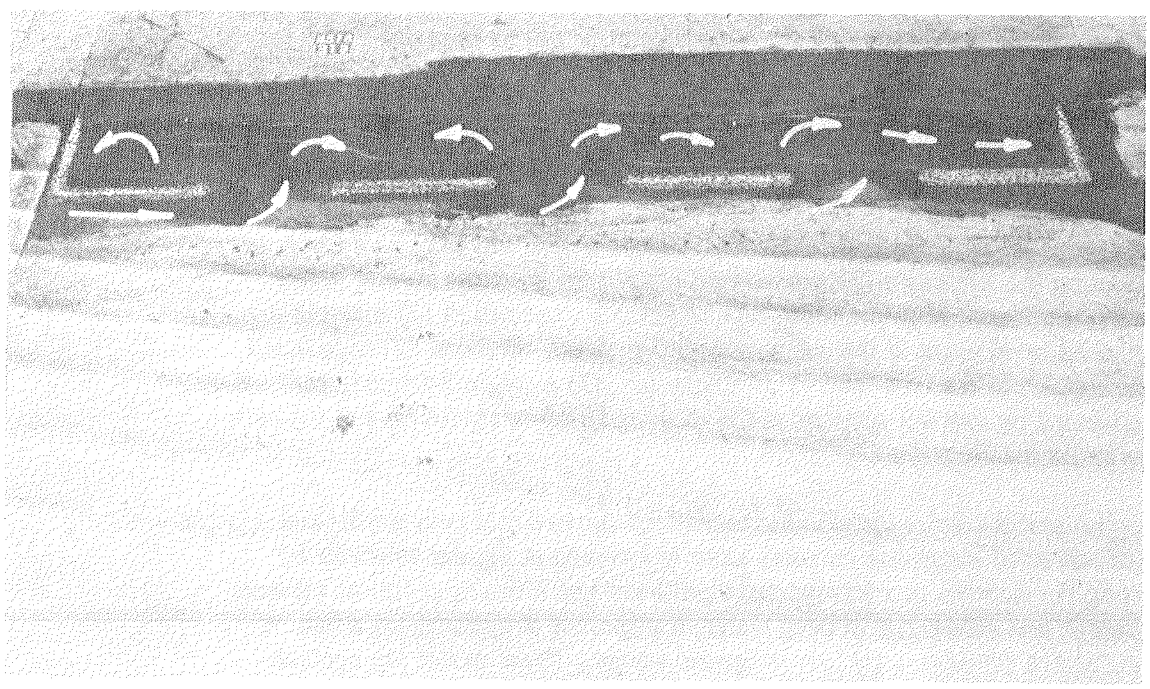


Figure 43. Oceanside Beach model test results for detached segmented breakwater system with groins. Arrows indicate current direction (Curren and Chatham 1980)

Testing segmented breakwater plans at the -4.6-m depth contour (Figure 44) showed that shorter segments with shorter gaps produced weaker rip currents and retained most of the tracer material, but a large volume of stone was required for construction. Submerged structures at the -3.0-m depth revealed that breaking waves piled water between the breakwater and shoreline, and the seaward return of the water created strong rip currents and the loss of tracer material to deep water. Test results with segmented breakwaters located at the -1.5-m contour with gaps indicated there was too much wave transmission in the structure lee. Low sills were placed in the gaps (Figure 45) and were successful in retaining all but small quantities of tracer, and thus the low sills between breakwater segments appeared to reduce the total wave transmission and caused the least impact on longshore transport.

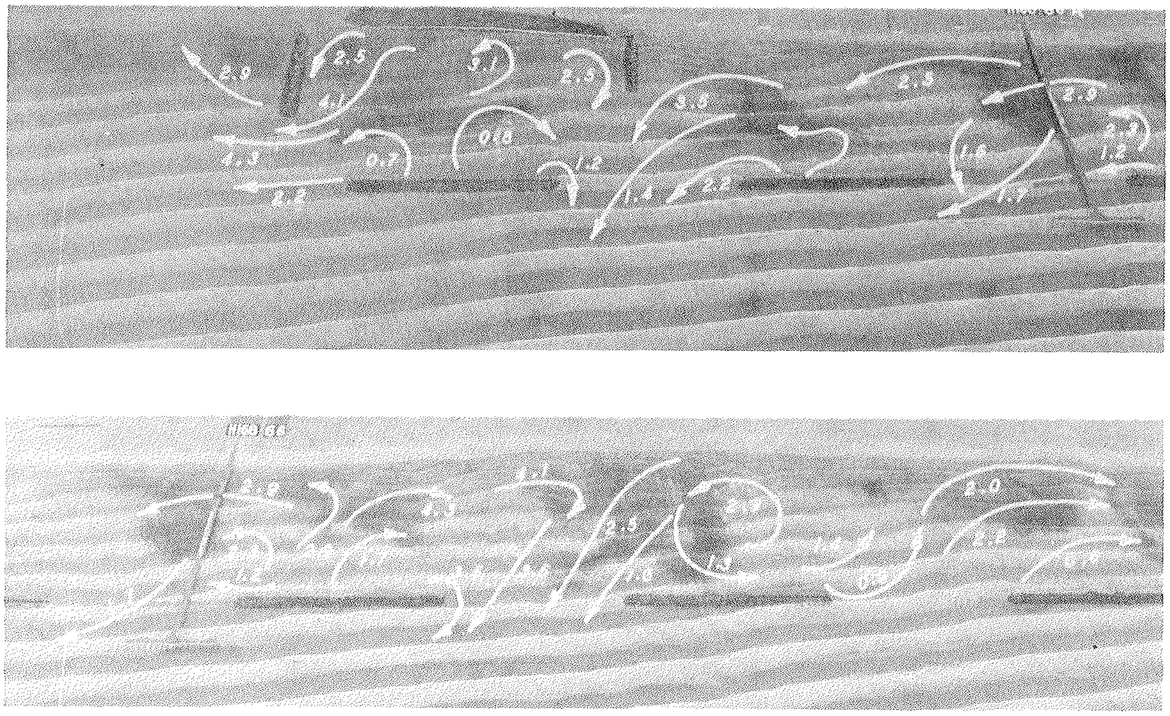


Figure 44. Typical wave and current patterns and current magnitudes for segmented detached breakwaters at the -4.6-m contour in the Imperial Beach model (Curren and Chatham 1977)

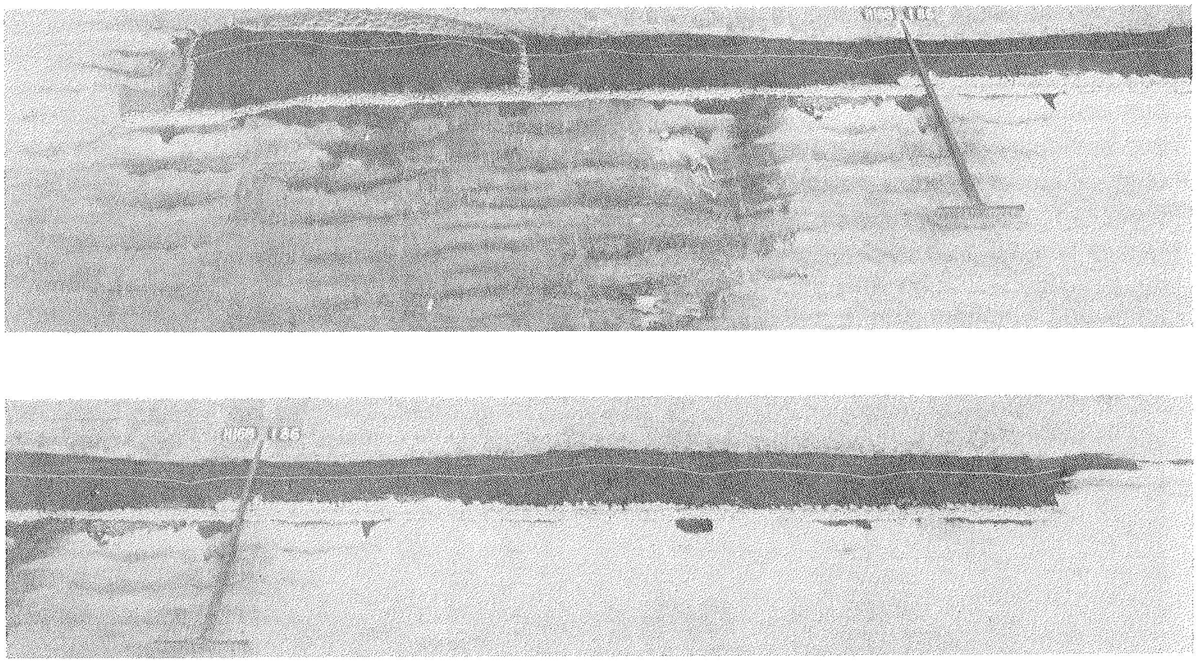


Figure 45. Results of Imperial Beach model study for a single detached breakwater with low sills at -1.5-m depth contour (Curren and Chatham 1977)

4 Structural Design Guidance

Structural Design Objectives

Traditional high-crested breakwaters with a multi-layered cross section are typically used for navigation purposes at entrance channels and harbors; but may not be appropriate for a structure used to protect and stabilize a beach, shoreline, or wetland. Adequate protection may be more economically provided by a low-crested or submerged structure composed of a homogeneous pile of stone. The greater tolerance of wave transmission at such sites has resulted in low-crested rubble-mound breakwaters being widely used or considered for use in beach stabilization, shore protection, and wetland development or protection. Recent laboratory tests specifically related to low-crested structures have resulted in empirical relationships to determine both the stability and performance characteristics of low-crested rubble-mound structures. The focus of this chapter of the report is aimed at the structural aspects of rubble-mound breakwaters.

The main structural design objectives of detached breakwaters are to ensure that the structure remains stable and provides acceptable performance characteristics throughout the project design life. Low-crested breakwater design consists of determining the required crest elevation, crest width, structure slope, and armor requirements to provide the desired stability and functional performance under the anticipated design wave and water level conditions. Structural guidance is provided to aid in the development of a breakwater cross section to meet both the functional and structural needs of a given project location.

Design Wave and Water Level Selection

In the selection of design water levels and design waves for a project, the conditions critical to structure stability and performance must be considered. The conditions represent critical threshold combinations of tide level, surge level, wave conditions, etc., which, if surpassed, will endanger the project and/or make the structure nonfunctional during their occurrence

(EM 1110-2-1414). Methods to estimate the probabilities of exceedence of such critical conditions along with detailed guidance on the determination and selection of water levels and wave heights for coastal engineering design are presented in EM 1110-2-1414, *Water Levels and Wave Heights for Coastal Engineering Design*. Such critical conditions may be tolerable on a more frequent basis for shore stabilization structures compared to navigation structures, since such structures are primarily used to prevent erosion, not to protect people's lives. A decrease in design level may also offer a substantial cost savings over the traditional SPM design approach. These factors need to be considered when selecting design water levels and wave heights for detached breakwaters used as shoreline stabilization structures.

Water levels

The entire range of possible water levels is needed for the structural design of beach stabilization structures. High-water levels are used to estimate maximum depth-limited breaking wave heights and to determine crest elevations. Low-water levels are generally used for toe protection design. Water levels can be affected by astronomical tides, storm surges, seiches, river discharges, natural lake fluctuations, and reservoir storage limits.

Design water levels are typically described statistically in terms of frequency, or probability that a given water level will be equalled or exceeded, or its return period in years. Thus, for example, the water level that is exceeded once in 50 years (a 2-percent probability of being exceeded in any 1 year) might be specified as a design water level. Significant deviations from predicted astronomical tide levels will occur during storms because of meteorological tides (storm surges) caused by strong onshore winds and low atmospheric pressure. Consequently, design water levels for a structure may include a storm surge with a specified return period. Detailed information on the prediction of astronomical tides and storm surge are available in EM 1110-2-1414 and EM 1110-2-1412.

Waves

Wave data required for structural design differ from data needed for functional design. Structural design generally focuses on larger waves in the wave climate since these waves represent critical conditions which may endanger the structure's stability. Structural stability criteria are most often stated in terms of extreme conditions which a coastal structure must survive without sustaining significant damage. The conditions usually include wave conditions of some infrequent recurrence interval, say 25 or 50 years.

Wave height statistics to determine design conditions will normally be based on hindcast wave data since a relatively long record is needed for extrapolation and wave gauge records rarely cover a sufficient duration. WIS has developed hindcast data spanning 20 years for all three ocean coasts and the Great Lakes. Hindcast data are normally presented for relatively deepwater conditions. Since detached breakwaters are placed in the nearshore environment, the selected design wave height must be analytically propagated shoreward to the structure. The deepwater significant wave height and significant or peak spectral wave period can be used along with water level and bathymetric data to perform refraction and shoaling analyses which determine wave conditions at the site. Several numerical models are available to perform these operations and are presented as part of the CMS (Cialone et al. 1992).

The choice of design wave conditions for structural stability should consider whether the structure is subjected to the attack of nonbreaking, breaking, or broken waves. Wave conditions at a structure site depend critically on the existing water level. Consequently, a design still-water level (swl) or range of water levels must be established in determining wave forces on a structure. Structures may be subjected to radically different types of wave action as the water level at the site varies. A given structure might be subjected to nonbreaking, breaking, and broken waves during different stages of a tidal cycle. Critical design conditions are the wave and water level combinations which result in maximum forces or minimum structural stability.

Selection of design wave heights for nearshore structures will often be controlled by depth-limited waves. The depth-limited breaking wave height for the given design water level should be calculated and compared with the unbroken design storm wave height, and the lesser of the two chosen as the design wave. Maximum depth-limited breaking wave heights can be estimated using procedures found in Chapter 7 of the *Shore Protection Manual* (1984).

If breaking in shallow water does not limit wave height, a nonbreaking wave condition exists. For nonbreaking waves, the design height is selected from a statistical wave height distribution. The selected design height depends on whether the structure is defined as *rigid*, *semirigid*, or *flexible* (*Shore Protection Manual* 1984). For *rigid* structures, such as cantilever steel sheet-pile walls, where a large wave within the wave train can cause failure of the entire structure, the design wave height is normally based on H_1 (=1.67 H_s, the average of highest 1 percent of all waves). For *semirigid* structures, the design wave height can range from H_{10} (=1.27 H_s, the average of highest 10 percent of all waves) to H_1. Steel sheet-pile cell structures are semirigid and can absorb wave pounding; therefore a design wave of H_{10} may be used. For *flexible* structures, such as rubble-mound structures, the design wave height H_{10} is typically used. Waves higher than the design wave height impinging on flexible structures seldom create serious damage for short durations of extreme wave action.

Damage to rubble-mound structures is usually progressive, and an extended period of destructive wave action (waves greater than design conditions) is required before a structure ceases to provide protection. It is therefore necessary in selecting a design wave to consider both the frequency of occurrence of damaging waves and economics of initial construction, protection, and maintenance. On the Atlantic and Gulf coasts of the United States, hurricanes may provide the design criteria. The frequency of occurrence of the design hurricane at any site may range from once in 20 years to once in 100 years. On the North Pacific Coast of the United States, the weather pattern is more uniform and severe storms are likely each year. The use of H_s as a design height under these conditions could result in significant annual damage due to a frequency and duration of waves greater than H_s in the spectrum. Higher wave heights such as H_{10} or H_5 may be advisable to reduce maintenance costs (*Shore Protection Manual* 1984).

Structural Stability

Structural stability analyses are performed to determine required armor units or to predict expected level of damage that will occur for a given structure exposed to selected design wave and water level conditions. Structural stability can be divided into two types: static and dynamic. Conventional breakwaters have been designed to remain statically stable or allow zero damage to rigid and semirigid structures and less than 5 percent damage to rubble-mound structures for wave conditions not exceeding design conditions. Recent efforts (Ahrens 1987,1989; Van der Meer 1990, 1991; Sheppard and Hearn 1989) have focused on the design of dynamically stable structures such as reef breakwaters where initial crest heights are allowed to be reshaped due to wave attack. The stability of such structures is measured in terms of reduction in crest height due to wave attack.

The stability of a rubble-mound structure can be influenced by a number of parameters including wave and water level conditions, armor characteristics (size, shape, placement methods, etc.), crest elevation and width, structure slope, and overall structure permeability. A number of dimensionless parameters including the stability coefficient, stability number, and spectral stability number have been developed by various researchers (*Shore Protection Manual* 1984; Ahrens 1984,1987) to incorporate the influence of environmental variables and structural design parameters into a single parameter. Such parameters are useful in analyzing the influence of each variable on the overall stability of the structure.

Stabilities of three different types of rubble-mound breakwaters are presented to aid in the design of a nearshore breakwater for shoreline protection. The three types are defined as conventional, statically stable low-crested, and dynamically stable reef breakwaters. Methods are presented for each structure type to assess the structure's stability and determine stone dimensions and crest elevations required to provide a stable structure.

Conventional breakwaters

Conventional rubble-mound breakwaters (Figure 46) have been designed for "zero" damage (less than 5 percent structural damage) under design wave conditions. In the case of offshore breakwaters, this usually means specifying the crest elevation such that little to no overtopping occurs, since the volume of water overtopping the crest has been found to be an important parameter in determining rear slope stability (Sheppard and Hearn 1989). Zero damage and minimal overtopping are two assumptions incorporated into the Hudson stability formula (*Shore Protection Manual* 1984),

$$W = \frac{w_r H^3}{K_D (S_r - 1)^3 \cot\theta} \tag{20}$$

where W is the weight of the individual armor unit; w_r is the unit weight of the armor unit; H is the design wave height; K_D is the stability coefficient; S_r is the specific gravity of the armor unit; and θ is the angle of structure slope measured from horizontal.

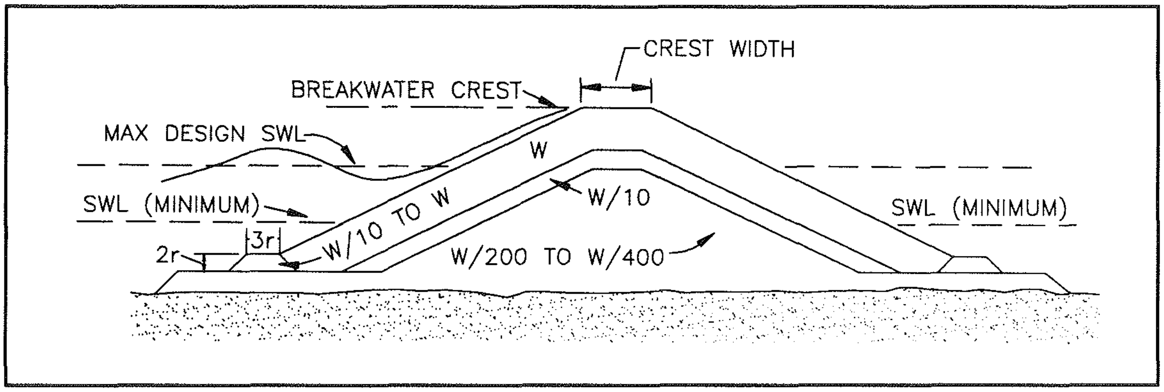


Figure 46. Cross section for conventional rubble-mound breakwater with moderate overtopping (*Shore Protection Manual* 1984)

The Hudson formula has been used extensively in the United States for breakwater design. However, apparent shortcomings of Hudson's formula, including lack of influence of wave period and the fact that it is based on regular wave tests, have been the subject of much discussion in recent years. Additional research aimed at such concerns has been conducted by a few investigators (Van der Meer 1987, Carver and Wright 1992).

Van der Meer (1987) derived two stability equations, one for plunging (breaking) waves and one for surging (nonbreaking) waves. These equations are as follows:

Plunging (breaking) waves

$$\frac{H_s}{\Delta\ D_{n50}} = 6.2\ P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} (\xi_z)^{-0.5} \tag{21}$$

Surging (nonbreaking) waves

$$\frac{H_s}{\Delta\ D_{n50}} = 1.0\ P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \sqrt{\cot\theta}\ \xi_z^P \tag{22}$$

where ξ_z is the surf similarity parameter,

$$\xi_z = \frac{\tan\theta}{\sqrt{\dfrac{2\pi\ H_s}{g\ T_z^2}}} \tag{23}$$

Δ	= relative mass density of stone, = $\rho_a/\rho_w - 1$	(24)
ρ_a	= mass density of armor	
ρ_w	= mass density of water	
D_{n50}	= nominal diameter of stone, = $(W_{50}/\rho_a)^{1/3}$	(25)
W_{50}	= 50 percent value (median) of the mass distribution curve	
P	= permeability coefficient of the structure as defined by Van der Meer (1987) (Figure 47)	
S	= damage level, = $A_e\ /\ D_{n50}^{\ 2}$	(26)
A_e	= eroded cross-sectional area in profile	
N	= number of waves (storm duration)	
ξ_z	= surf similarity parameter	
T_z	= average wave period	

The term on the left side of Equations 21 and 22 is referred to as the stability number N_s as defined by Van der Meer.

$$N_s = \frac{H_s}{\Delta\ D_{n50}} \tag{27}$$

Van der Meer's equations clearly include more explicit dependence on important parameters of the problem than Hudson's formula. They are formulated in terms of irregular wave parameters. A dependence on wave period comes in through the surf similarity parameter, ξ_z. Permeability, which has been shown to impact stability, is also included as well as damage level and storm duration. However, there are some important explanations and qualifications which need to be considered when applying Van der Meer's equations. Van der Meer's definitions of significant wave height, permeability, and acceptable damage levels must be used when applying the equations. Also, design conditions must fall within the acceptable ranges of structure slope, wave steepness, storm duration, and mass density. Both the Hudson formula and Van der Meer's equations are suitable in stability analysis

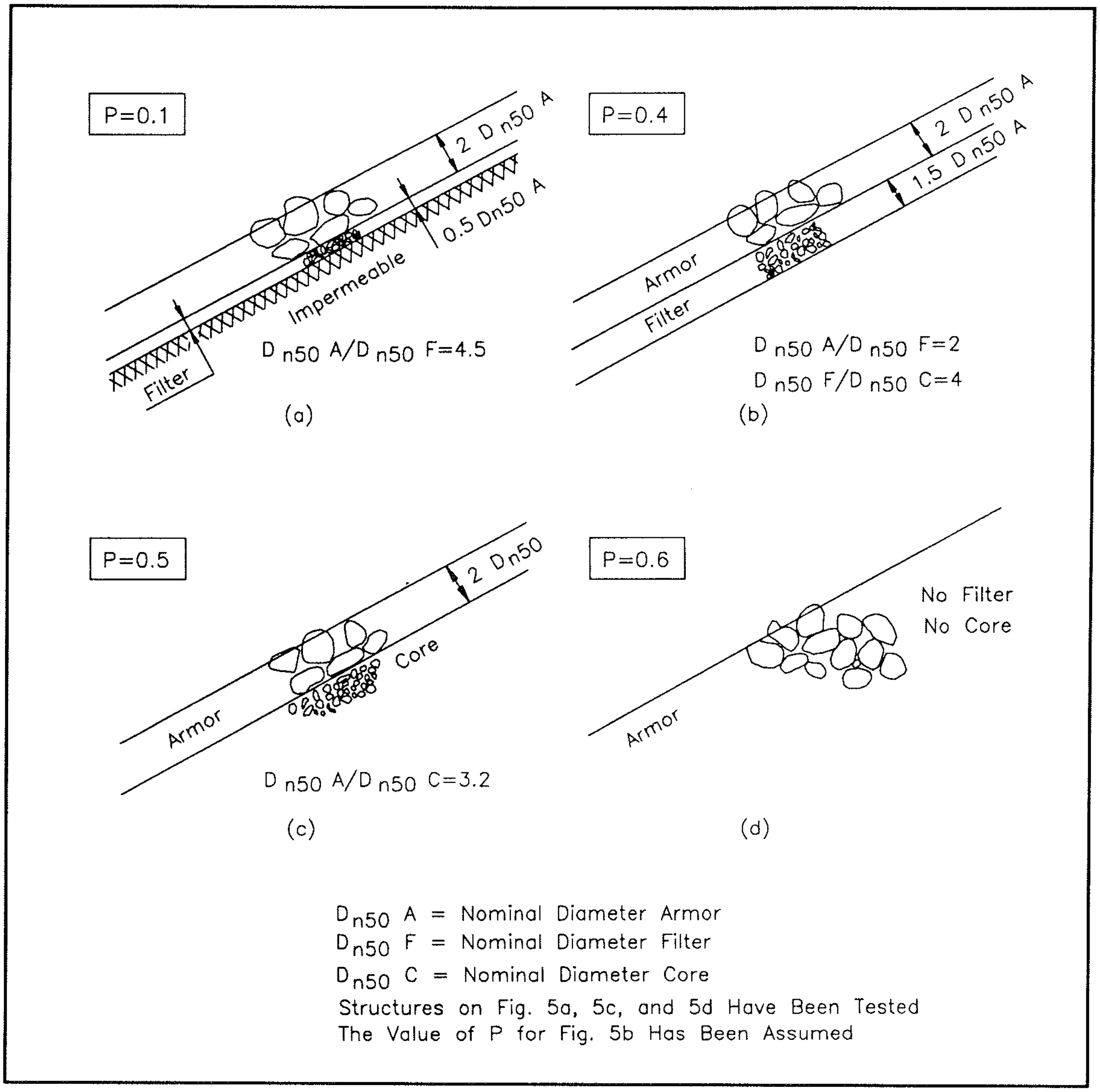


Figure 47. Permeability coefficient *P* (Van der Meer 1987)

of breakwater armor layers; however, each has limitations and assumptions which need to be considered in design.

Statically stable low-crested breakwaters

Low-crested breakwaters are similar to conventional non-overtopped structures, but are more stable due to the fact that a large part of the wave energy can pass over the breakwater. The increase in stability can be physically explained by the different wave motion which acts on the structure's slope. For non-overtopped structures, the majority of runup water will return during the down-rush (except for the part that penetrates the core).

With a lower crest the wave overtops the structure and the rundown will be much smaller, which increases stability. An example of a low-crested breakwater is shown in Figure 48.

The stability of a low-crested breakwater with the crest above the still-water level is first established as being a non-overtopped structure (Van der Meer 1990). Stability formulae such as Hudson's formula or Van der Meer's formulae can be used to determine the required stone diameter of the non-overtopped breakwater. Required stone diameter for an overtopped breakwater can then be determined by multiplying the stone diameter for a non-overtopped breakwater by a reduction factor to account for the increase in stability. After analysis of several data sets, Van der Meer (1991) describes the increase in stability as a function of dimensionless freeboard R_p^* in the form of the following reduction factor:

Reduction factor for D_{n50}, $r = 1/(1.25 - 4.8\, R_p^*)$ (28)

for $0 < R_p^* < 0.052$

where R_p^* = dimensionless freeboard, $R_c/H_s\ (s_{op}/2\pi)^{0.5}$ (29)

R_c = crest freeboard, level of crest relative to still water

s_{op} = fictitious wave steepness, $2\pi H_s/gT_p^{\,2}$ (30)

T_p = peak wave period

Equation 28 describes the stability of a statically stable low-crested breakwater with the crest above still-water level in comparison with a non-overtopped structure. Figure 49 shows Equation 28 for various wave steepnesses. The reduction factor for the required stone diameter can be read off the graph or computed using Equation 28. It can be seen in Figure 49 that an average reduction of 0.8 in diameter is obtained for a structure with the crest height at the still-water level. The required mass is a factor $(0.8)^3 = 0.51$ of that required for a non-overtopped structure.

Dynamically stable reef-type breakwaters

A reef breakwater is a low-crested rubble-mound breakwater without the traditional multi-layer cross section (Figure 50). This type of breakwater is little more than a homogeneous pile of stones with individual stone weights similar to those used in the armor and first underlayer of conventional breakwaters (Ahrens 1989). Because of their high porosity and low crest, reef breakwaters are stable to wave attack and, at the same time, if they are high enough, can dissipate wave energy effectively. Since they have no core, they cannot fail catastrophically and therefore a logical strategy is to allow them to adjust and deform to some equilibrium condition (Ahrens 1989). The equilibrium crest height, along with corresponding transmission, are the main design parameters. Tolerable crest height reductions and maintenance requirements should be defined by the designer.

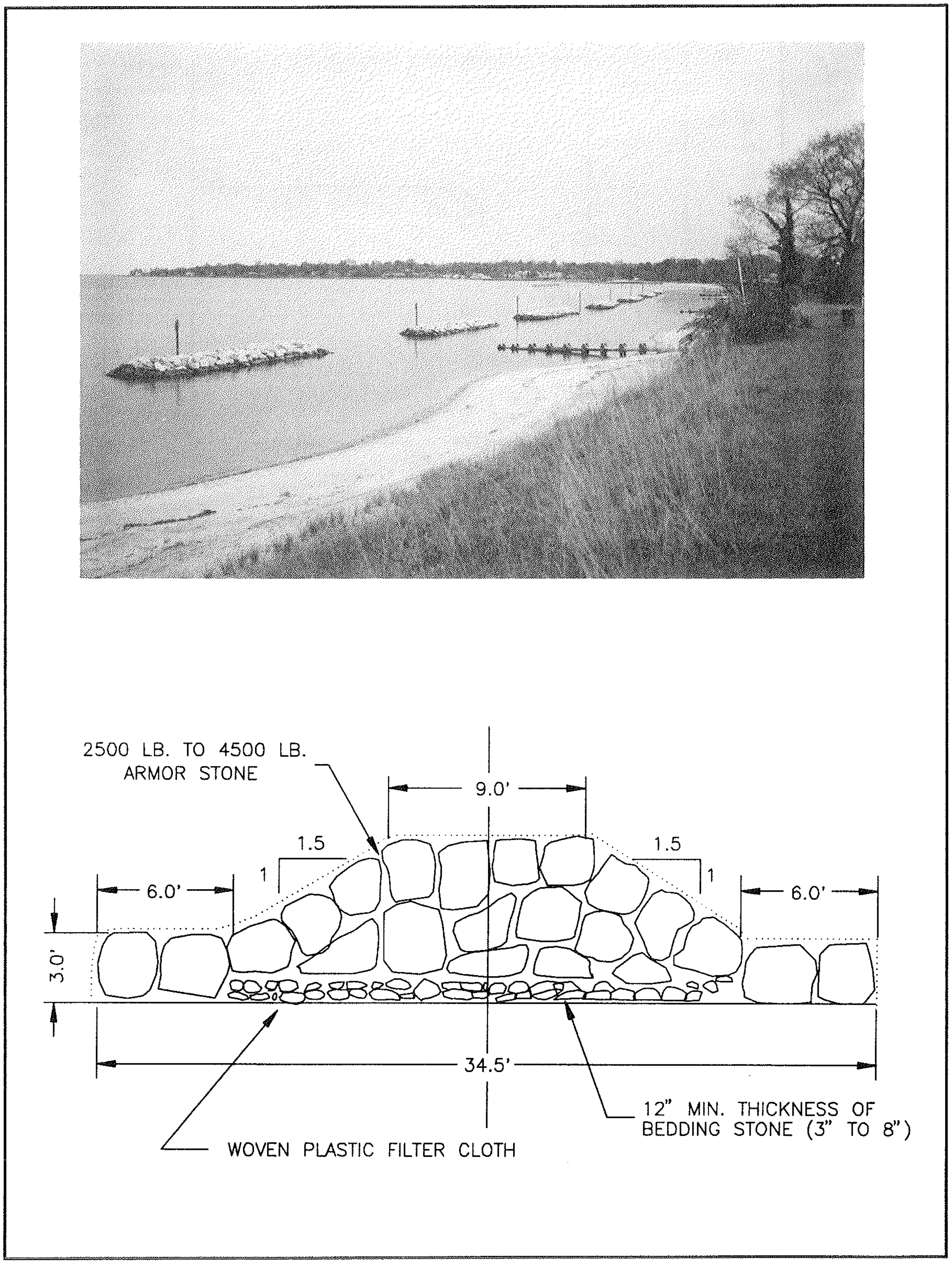


Figure 48. Example of a low-crested breakwater at Anne Arundel County, Maryland (Fulford and Usab 1992)

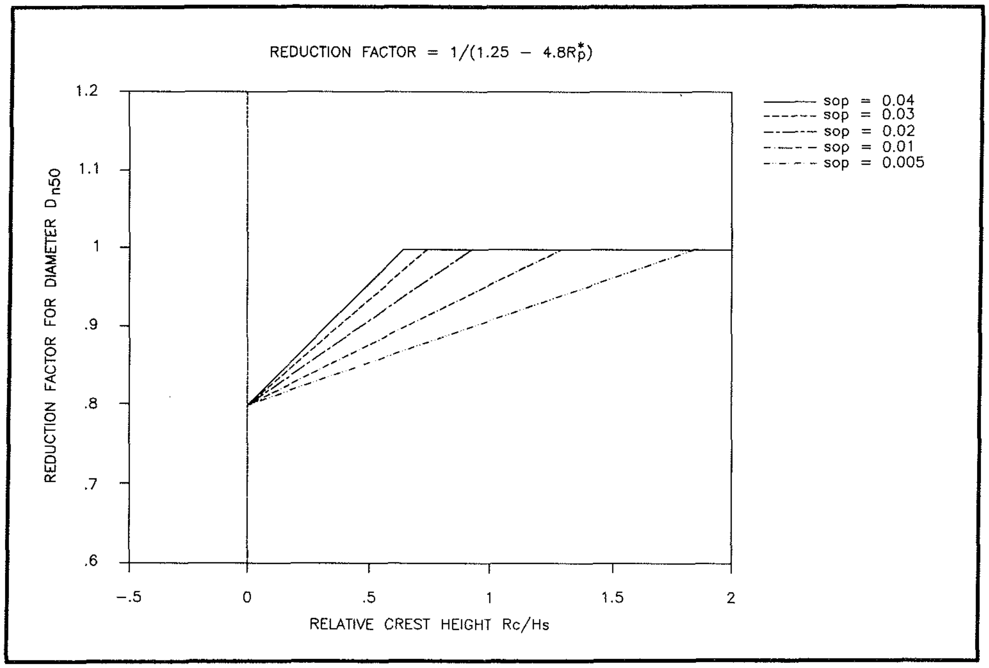


Figure 49. Design graph with reduction factor for the stone diameter of a low-crested structure as a function of relative crest height and wave steepness (Van der Meer 1991)

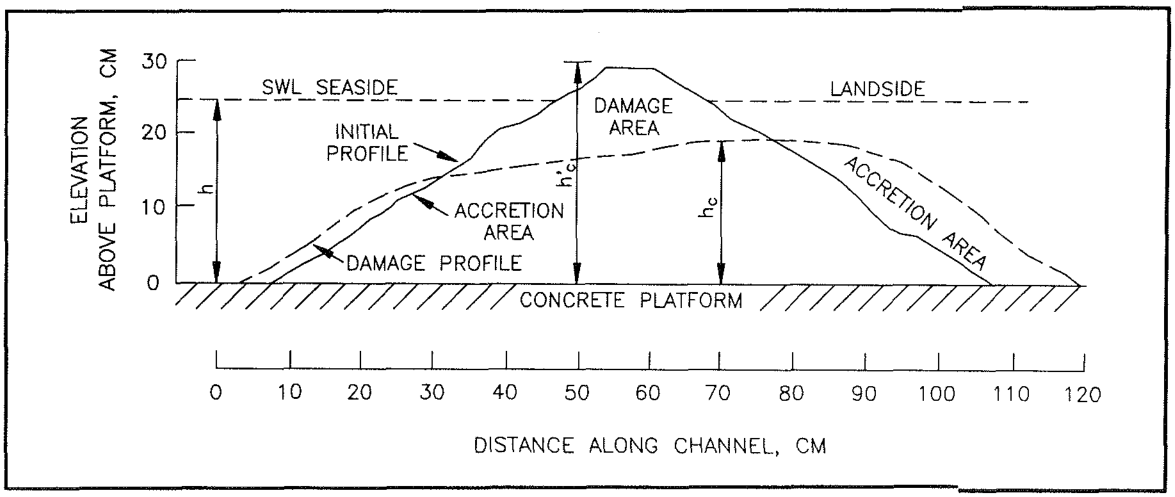


Figure 50. Typical reef profile, as built, and after adjustment to severe wave conditions (Ahrens 1987)

The analyses on stability of reef breakwaters by Ahrens (1987,1989) and Van der Meer (1990) concentrated on change in crest height due to wave attack. Ahrens defined a number of dimensionless parameters used in describing behavior of the structure. The main parameter is the crest height reduction factor h_c/h_c', which is the ratio of the crest height at the end of the laboratory test h_c to the initial crest height h_c'.

Another important parameter defined by Ahrens is the spectral (or modified) stability number N_s^* defined as:

$$N_s^* = \frac{H_{mo}^{2/3}\, L_p^{1/3}}{\Delta D_{n50}} \tag{31}$$

where H_{mo} is the zero-moment wave height of the incident wave spectrum and L_p is the Airy wavelength calculated using the peak wave period T_p and the local water depth h at the toe of the structure.

The reduced crest height of a reef breakwater is estimated by:

$$h_c = \sqrt{A_t \,/\, a \, \exp(N_s^*)} \tag{32}$$

where "a" is a coefficient and A_t is the structure's cross-sectional area.

Ahrens (1989) gives several equations for the coefficient "a". Van der Meer (1988) tested several structures with crest heights, water depths, bulk numbers, and slope angles different than Ahrens. Van der Meer (1990) reanalyzed the data of Ahrens (1987) and Van der Meer (1988), and derived a new equation for the coefficient "a." The resulting equation is similar to Ahrens, but valid for a wider range of conditions. The resulting equation for the coefficient "a" is given by:

$$"a" = -0.028 + 0.045C' + 0.034\, h_c'/h - 6x10^{-9}\, B_n^2 \tag{33}$$

and $h_c = h_c'$ if h_c in Equation 32 $> h_c'$.

where

C' = average structure slope "as built" (normal range: $1.5 \leq C' \leq 3.0$)

B_n = bulk number, $A_t \,/\, D_{n50}^2$ (34)

A_t = $B\, h_c' + C'\, h_c'^2$ (35)

B = crest width (normally taken as 3 median stones wide, $3\ D_{n50}$)

Crest height reduction of a reef breakwater as shown in Figure 50 can be calculated using Equations 32 and 33. Design curves can also be produced from these equations which give the crest height as a function of H_s (Figure 51) or even N_s^* (Figure 52) for a given water level, structure slope, initial crest height, and bulk number. Bulk number can be described as the equivalent number of median stones per median stone width in the breakwater cross section. The reefs tested by Ahrens and Van der Meer have relatively high bulk numbers (B_n greater than 200) compared to many structures that are actually being built in the United States. Therefore, bulk numbers for a given design should be checked against the valid ranges of the above equations to assure accurate results.

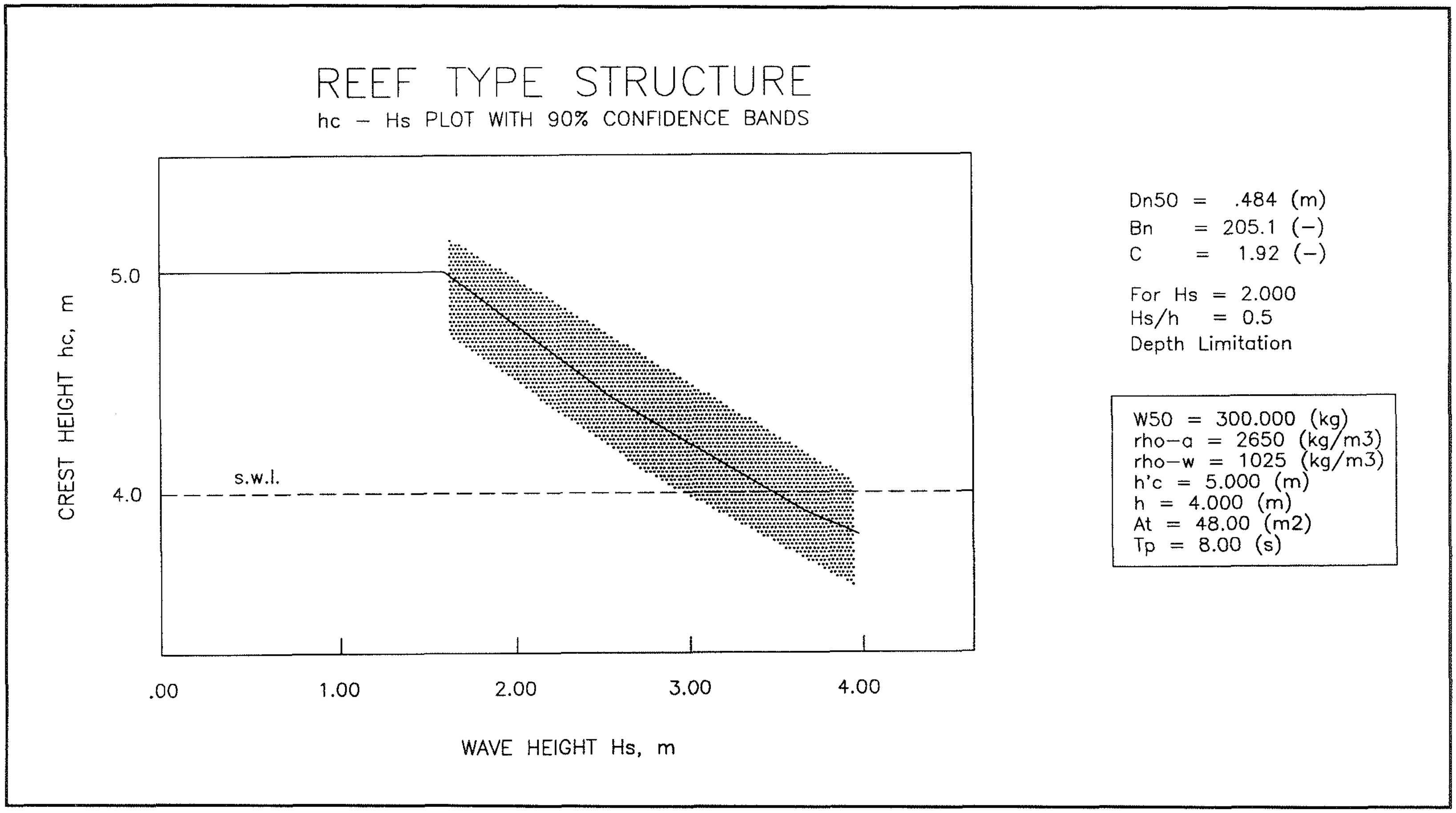


Figure 51. Design graph of a reef type breakwater using H_s (Van der Meer 1991)

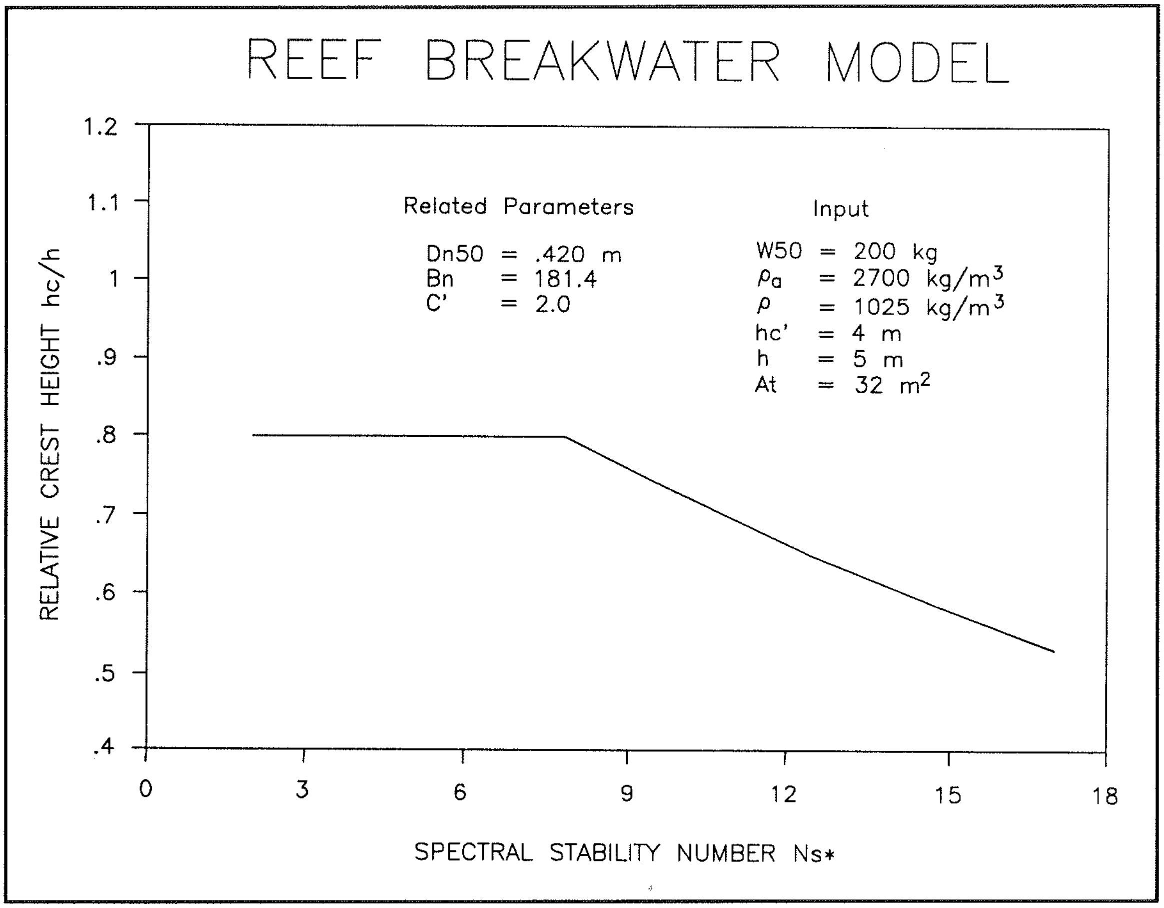


Figure 52. Design graph of reef type breakwater using the spectral stability number N^*_s (Van der Meer 1990)

Performance Characteristics

Low-crested rubble-mound breakwaters offer an attractive alternative to the protection of shorelines against direct wave attack. It is important, both functionally and structurally, to assess the effectiveness of a given breakwater design by predicting the amount of wave energy transmitted, reflected, and dissipated by the structure. Such performance characteristics involve a number of complex processes. Some incident wave energy may be reflected by the structure, some wave energy may be dissipated by turbulent interaction with the armor layer, some wave energy may be dissipated internally within the core of the permeable structure, and some may be transmitted through or over the structure resulting in wave action in the lee of the structure. Important factors identifiable in the process include incident wave conditions and the structure's shape, material composition, and degree of emergence or submergence. Figure 53 shows some of the key parameters involved in determining a breakwater's performance characteristics.

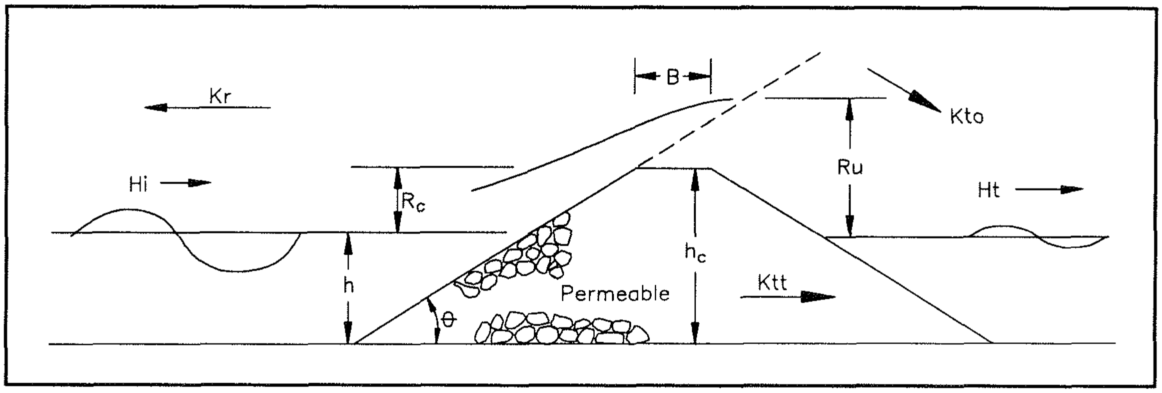


Figure 53. Terminology involved in performance characteristics of low-crested breakwaters

Transmission

Transmission of wave energy beyond rubble structures has been studied by many researchers covering a wide variety of structures and resulting in numerous methodologies and equations useful in predicting the characteristics of transmitted waves. The majority of breakwaters used for shoreline stabilization consist of low-crested permeable structures which have wave energy transmitted both through and over the structure. Three methods or procedures applicable to such structures are presented below to aid the designer in determining a transmission coefficient K_t to be used in functional design. Each method was developed for a different range of structural and incident wave conditions. The designer must determine which method is most applicable.

The transmission coefficient K_t is generally defined as the ratio of the transmitted wave height to the incident wave height.

$$K_t = \frac{H_t}{H_i} \tag{36}$$

where

H_t = transmitted wave height
H_i = incident wave height

As stated previously, two types of wave transmission occur with low-crested permeable structures: wave regeneration caused by overtopping of the structure's crest, and wave energy transmitted through the permeable structure. Seelig (1980) approached the transmission problem by making independent estimates of energy transmitted by each condition and combining the two components to obtain the total transmitted energy:

$$K_t = \sqrt{(K_{to})^2 + (K_{tt})^2} \tag{37}$$

where

K_t = total wave transmission coefficient
K_{to} = overtopping transmission coefficient
K_{tt} = through transmission coefficient

This method was programmed as one of the modules in the Automated Coastal Engineering System (ACES) (Leenknecht, Szuwalski, and Sherlock 1993) titled "Wave Transmission Through Permeable Structures." Seelig's approach provides a method to estimate wave transmission for a wide range of structure types and geometry and for a wide range of wave conditions.

Ahrens (1987) developed a method to estimate wave transmission based on about 200 laboratory tests of reef breakwaters. Irregular wave tests were performed on both submerged and nonsubmerged reefs. Ahrens' approach is based on the use of two formulas which are selected depending on the relative freeboard (R_c/H_{mo}) value.

For relatively high reefs, $R_c/H_{mo} > 1.0$, the dominant mode is transmission through the reef. The transmission coefficient is largely a function of one variable which is the product of wave steepness and the bulk number.

$$K_t = \frac{1.0}{1.0 + \left(\frac{H_{mo}\, A_t}{L_p\, D_{n50}^2}\right)^{0.592}} \tag{38}$$

When the dominant modes of transmission result from wave overtopping or waves propagating over the crest of a submerged reef ($R_c/H_{mo} < 1.04$), a rather complex relation involving several variables is required to predict transmission coefficients.

$$K_t = \frac{1.0}{1.0 + \left(\frac{h_c}{h}\right)^{1.188} \left(\frac{A_t}{h\, L_p}\right)^{0.261} \exp\left[0.529\left(\frac{R_c}{H_{mo}}\right) + 0.00551\left(\frac{A_t^{3/2}}{D_{n50}^2\, L_p}\right)\right]} \tag{39}$$

It should be noted that Ahrens does not use the traditional definition of the transmission coefficient involving the incident wave height at the toe of the structure. A transmission coefficient, which is the ratio of the transmitted height to the height which would be measured at the same location in absence of the reef, is preferred since it eliminates loss of energy due to wave breaking which would have occurred if the structure were not present (Ahrens and Cox 1990). It is this type of coefficient predicted using Ahrens' equations which may cause them to be slightly higher than traditional coefficients.

Van der Meer (1991) developed a new formula for wave transmission at low-crested structures. After re-analyzing several data sets involving transmission at low-crested breakwaters, including Ahrens (1987) and recent tests by Daemen (1991), Van der Meer assumed that a linear relationship between the transmission coefficient K_t and the relative crest height R_c/D_{n50} is valid between minimum and maximum values of K_t. Figure 54 shows the basic graph for wave transmission. The linearly increasing curves are presented by:

$$K_t = a\, R_c \,/\, D_{n50} + b \tag{40}$$

with:

$$a = 0.031\, H_i \,/\, D_{n50} - 0.24 \tag{41}$$

Equation 41 is applicable to both conventional and reef breakwaters. The coefficient "b" for conventional breakwaters is given by:

$$b = -5.42\, s_{op} + 0.0323\, H_i/D_{n50} - 0.0017\, (B/D_{n50})^{1.84} + 0.51 \tag{42}$$

and for reef breakwaters by:

$$b = -2.6\, s_{op} - 0.05\, H_i/D_{n50} + 0.85 \tag{43}$$

Based on the results of all tests analyzed (Van der Meer 1991), the following minimum and maximum K_t values were derived. The minimum and maximum K_t values for conventional breakwaters are 0.075 and 0.75, respectively. For reef-type breakwaters, the minimum and maximum K_t values are 0.15 and 0.60, respectively.

The validity of the wave transmission formula (Equation 40) corresponds with the ranges of wave steepness and relative wave height tested. The formula is valid for:

$$1 < H_i/D_{n50} < 6 \quad and \quad 0.01 < s_{op} < 0.05$$

Both upper boundaries are physical bounds. Values of $H_i/D_{n50} > 6$ will cause instability of the structure and values of $s_{op} > 0.05$ will cause wave breaking on steepness. The lower boundaries are given for too low wave heights relative to rock diameter and for very low wave steepnesses. The formula may be applicable outside the range, but the reliability is low.

Reflection

Low-crested rubble-mound breakwaters, because of their high porosity, rough texture, and low profile, typically have low reflection coefficients. This is an advantage because it reduces the potential for toe scour, navigation problems, and erosion at nearby shorelines caused by reflected waves. The

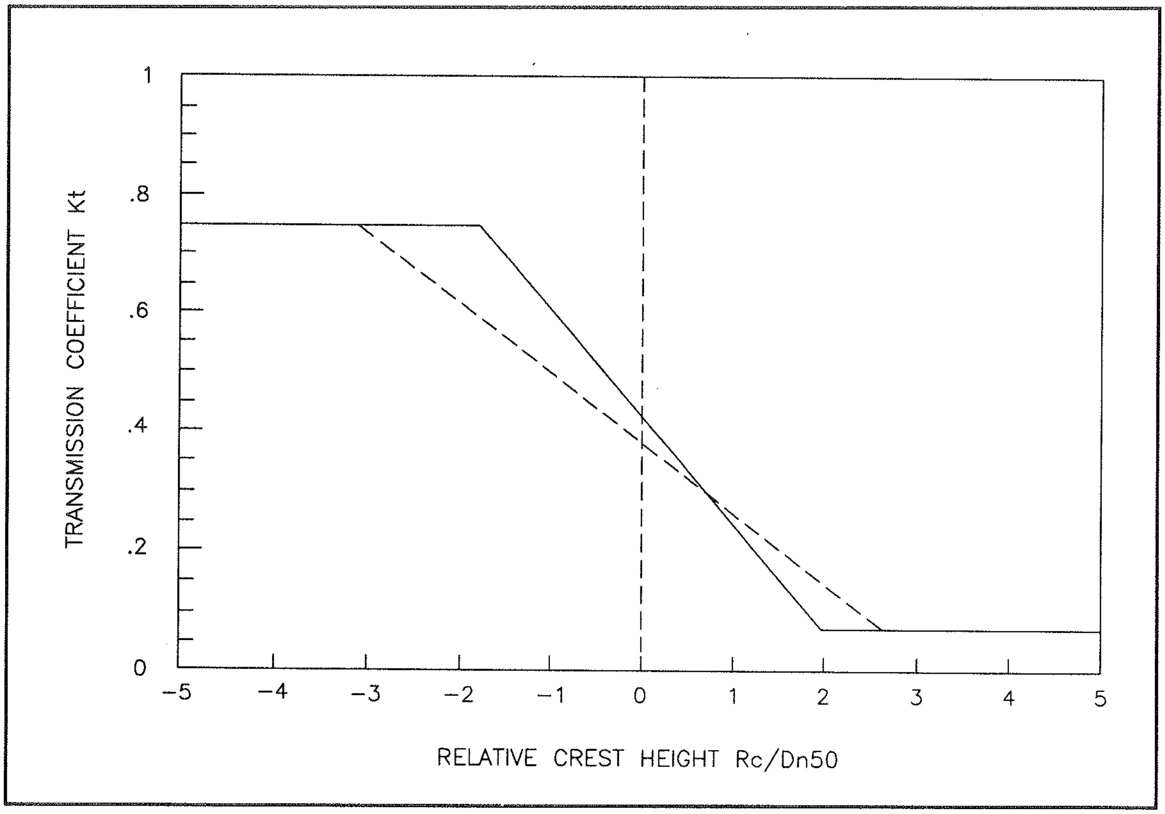


Figure 54. Basic graph for wave transmission versus relative crest height (Van der Meer 1991)

two most important variables that influence wave reflection from a low-crested structure are the relative depth, h/L_p, and the relative height, h_c/h (Ahrens and Cox 1990). Ahrens (1987) presents a formula for predicting the reflection coefficient for a reef breakwater.

$$K_r = \exp\left[C_1\left(\frac{h}{L_p}\right) + \frac{C_2}{\frac{h_c}{h}} + C_3\left(\frac{A_t}{h_c^2}\right) + C_4\left(\frac{R_c}{H_{mo}}\right)\right] \tag{44}$$

where

C_1 = -6.774
C_2 = -0.293
C_3 = -0.0860
C_4 = +0.0833

The ACES module "Wave Transmission Through Permeable Structures" also provides a method of determining a reflection coefficient. Other factors being equal, reflection coefficients increase with increasing wavelength and increasingly steeper slopes. Reflection coefficients also increase with increasing relative crest height, h_c/h, and increasing relative freeboard, R_c/H_{mo}, until the crest height reaches the upper limit of wave runup.

Energy dissipation

The ability of low and submerged rubble structures to dissipate wave energy has long been appreciated, but only in recent years has it been possible to quantify this property. There is not a lot of specific information available on the wave energy dissipating characteristics of rubble structures, even though this is regarded as one of the major advantages over other structure types (Ahrens and Cox 1990). The primary reason for this is that energy dissipation cannot be directly measured, but must be inferred from measurements of wave transmission and wave reflection. The basic conservation of energy for rubble structures can be written as:

$$K_t^2 + K_r^2 + \text{dissipation} = 1.0 \tag{45}$$

Ahrens used the prediction equations for transmission and reflection coefficients in the energy conservation relation given by Equation 45 to determine energy dissipation characteristics for given breakwater configurations. Figure 55 was developed by Ahrens (1987) to illustrate the distribution of wave energy in the vicinity of a reef breakwater. Generally, the greatest energy dissipation was observed for short period waves on structures with crest heights high enough to be non-overtopped. The lowest energy dissipation of about 30 percent occurred for reefs with a relative crest height less than 0.7 ($h_c/h < 0.7$). For submerged reefs, energy dissipation increases with increasing steepness H_{mo}/L_p and with increasing relative reef width A_t/hL_p. Structures with crests near the still-water level will dissipate between 35 and 70 percent of incident wave energy, with dissipation being strongly dependent on relative reef width. For structures with moderate to heavy overtopping ($0 < R_c/H_{mo} < 1.0$), energy dissipation is strongly dependent on relative reef width, but not on wave steepness.

Detailing Structure Cross Section

Coastal structures must be designed to satisfy a number of sometimes conflicting design criteria, including structural stability, functional performance, environmental impact, life-cycle costs, and other constraints which add challenge to the designer's task (*Shore Protection Manual* 1984). The requirement to satisfy a number of different design criteria often results in the designer performing a number of iterative analyses to assure that the selected cross-section provides the desired functional performance and structural stability at the least cost over the design life of the project. Optimization of rubble-mound breakwaters is discussed in Chapter 5.

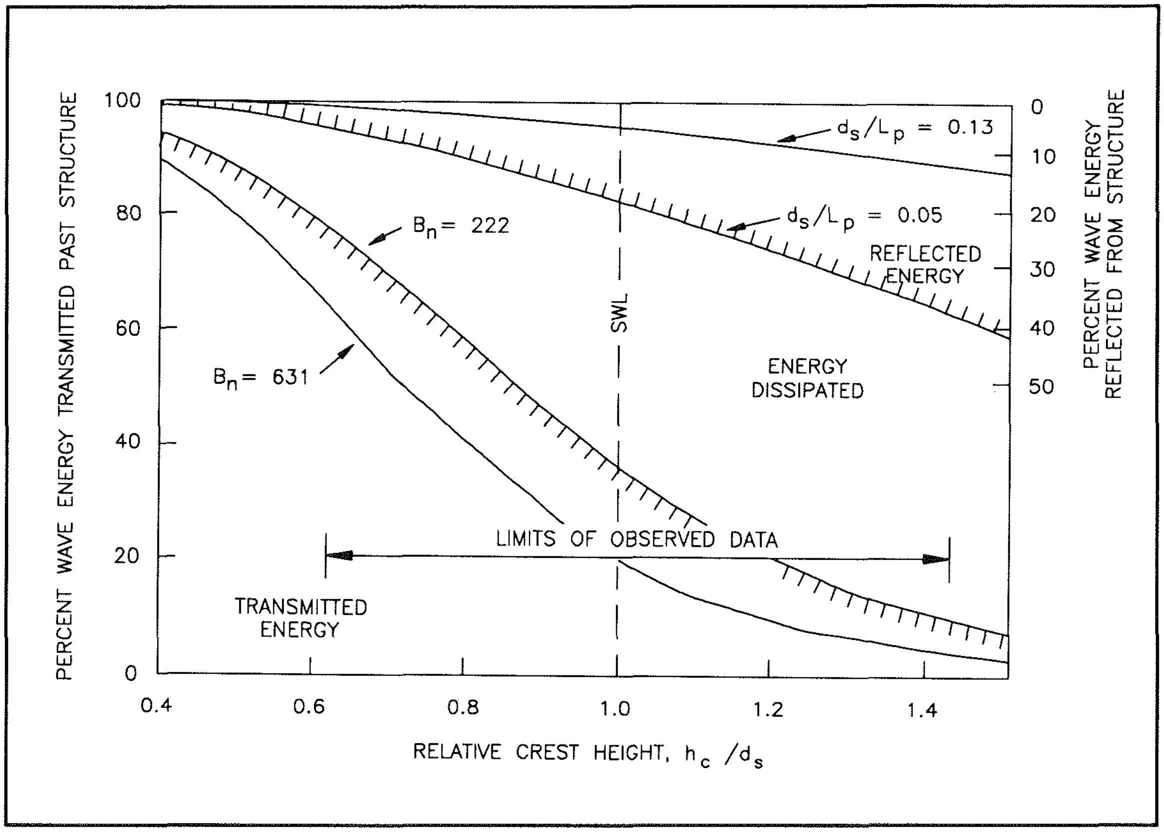


Figure 55. Distribution of wave energy in the vicinity of a reef breakwater (Ahrens 1987)

A conventional rubble-mound structure is normally composed of a bedding layer and a core of quarrystone covered by one or more layers of larger stone and an exterior layer of large quarrystone or concrete armor units. Figure 46 shows a typical rubble-mound section for high wave energy environments where moderate overtopping conditions are expected. The traditional multi-layer design may not be required or constructable for projects located in lower wave energy environments or shallow water. Geometry places some serious constraints in shallow water, where it is difficult to include all theproper layers, proper thickness, proper stone weight, etc. when the structure is only 4 ft high. Reef breakwaters have recently become more widely used as beach stabilization structures. This type of breakwater is little more than a homogeneous pile of stones placed on a bedding or filter layer. Figures 56 and 57 show cross sections of existing reef breakwater projects.

Developing a breakwater cross section consists of determining the required crest elevation, crest width, structure slope, armor requirements, and bedding layer requirements to provide the desired stability and functional performance characteristics under anticipated design wave and water level conditions. General design guidance used to develop the cross section of a conventional rubble-mound breakwater can be found in Chapter 7 of the *Shore Protection Manual* and in Chapter 4 of EM 1110-2-2904, *Design of Breakwaters and*

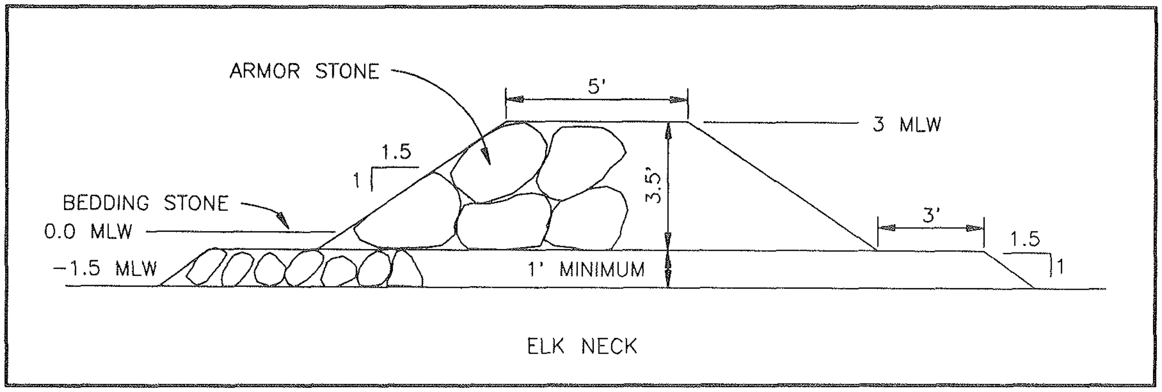


Figure 56. Cross section of reef breakwater at Redington Shores at Pinnelas County, Florida (Ahrens and Cox 1990)

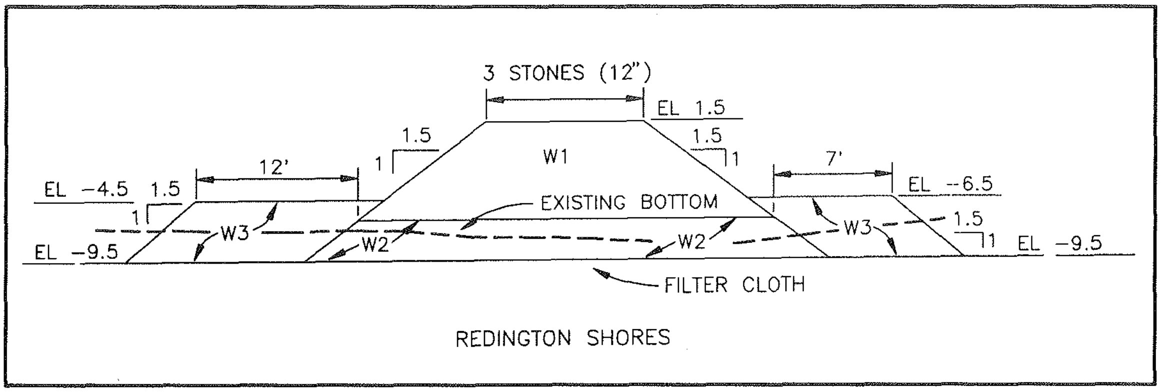


Figure 57. Cross section of reef breakwater at Elk Neck State Park, Maryland (Ahrens and Cox 1990)

Jetties. The design and construction of low-crested breakwaters, including reef breakwaters, uses similar procedures to those specified in the above manuals, but involves different design guidance for several steps in the procedure.

Crest height, crest width, and structure slope

Iterative analyses involving the assessment of a range of crest elevations, crest widths, and structure slopes are required to determine the influence of each on both stability and functional performance and ultimately develop the optimum cross section. The crest elevation of a low-crested breakwater is one of the most critical parameters in the cross sectional design due to the considerable influence of crest elevation on both structural stability and functional performance. Small changes in crest elevation can result in significant changes in stability and wave transmission characteristics. The crest width and structure slope also influence stability and performance of the structure; however, less dramatically than crest elevation. Therefore, these parameters often follow guidance and ranges used for conventional structures.

Shore Protection Manual guidance suggests a minimum crest width equal to the combined widths of three armor units. Structure slopes normally range from 1V:1.5H to 1V:3H. Selection of crest elevation can be performed using guidance previously discussed in the "Structural Stability" and "Performance Characteristics" sections. The influence of crest elevation on stability of low-crested breakwaters can be estimated using Figure 49 for statically stable low-crested breakwaters and Equations 32 and 33 for reef breakwaters. A series of design curves for reef breakwaters similar to Figures 51 and 52 can be developed to aid in converging on the optimum design. The stability analysis will yield a relationship between crest elevation and armor unit requirements. The performance characteristics for each cross section can be computed using methods previously discussed. Overall analysis of each cross section's stability, performance characteristics, and costs will result in selection of the optimum cross section.

Armor gradation

Generally, reef breakwaters have been designed using stone gradations wider than ordinarily used for armor in conventional, multilayered breakwaters, as discussed in the *Shore Protection Manual* (1984). The advantages of a wide gradation is that it uses a larger portion of the stone produced by a quarry and therefore may be more economical. A wider gradation also makes it easier to satisfy the filter criteria that will be discussed in the following section. Gradation is easily represented in terms of median weight of armor stone W_{50} determined from stability analyses discussed previously. W_{50} is used to normalize the other percentile weight stones, i.e.,

$$W_x' = \frac{W_x}{W_{50}} \tag{46}$$

where x indicates the percentile of armor stone less than the given weight. For example, W_{15}' represents the ratio of W_{15} to W_{50}, where W_{15} is the stone size exceeding only 15 percent of all stones in the gradation.

Extensive studies of breakwater and riprap stability conducted in The Netherlands have produced two well-defined stone gradations (Van der Meer and Pilarczyk 1987), which are referred to as the Dutch wide and the Dutch narrow gradations. The wide gradation is defined by:

$$W_x' = [exp(0.01157x - 0.5785)]^3 \tag{47}$$

and the Dutch narrow gradation is defined by:

$$W_x' = [exp(0.003192x - 0.1597)]^3 \tag{48}$$

where x is entered as a percent to solve for various values of W_x'. The two Dutch gradations along with the gradation specified in the SPM (Ahrens 1975) are presented in Figure 58. The Dutch wide gradation is similar to well-graded riprap and the Dutch narrow gradation is similar to very uniform quarrystone. The two Dutch gradations can be used to provide upper and lower bounds for stone gradations used in low-crested breakwater design.

Bedding/filter layer considerations

Usually reef breakwaters are built on a bedding/filter layer. The bedding layer is designed to prevent excessive settlement of the structure due to armor stone sinking into the underlying sediment. The ratio of median armor stone size $D_{50}(A)$ to the median bedding stone size $D_{50}(B)$ provides a logical way to characterize the bedding size. Two methods are available to select the size of required bedding stone. Dutch guidance for revetment filter layers suggests that $D_{50}(A)/D_{50}(B)$ be approximately 4.5 or less (Van der Meer and Pilarczyk 1987). Ahrens (1975) suggests that $D_{15}(A)/D_{85}(B)$ should not be greater than 4.0 to ensure that the underlayer is not pulled out through the armor layer by wave action. Considering gradations used by Ahrens, a safe relation for median stone dimensions would be $D_{50}(A)/D_{50}(B)$ less than 6.8 (Ahrens and Cox 1990).

In low rubble-mound structures without a core, the bedding layer is often extended across the entire width of the structure and beyond the toe of the armor stone as shown in Figure 56. The bedding stone will often be subject to direct wave attack during low water levels. Bedding stones at the toe of the structure may not provide the desired stability or toe protection, resulting in additional stones required along the toe as shown in Figure 55. Stability against wave attack of exposed bedding stone is discussed in EM 1110-2-2904 and detailed guidance on toe protection can be found in the *Shore Protection Manual* (1984).

Geotextiles can be used beneath the bedding layer to improve foundation conditions or prevent the loss of sediment through the bedding layer if filter criteria between the bedding layer and underlying soil are not met. Filter criteria should be met between both the geotextile and bedding layer and the geotextile and underlying soil. Geotextiles are discussed in the *Shore Protection Manual* and by Moffatt and Nichol, Engineers (1983) and Eckert and Callender (1984), who present detailed requirements for using geotextile filters beneath quarrystone armor in coastal structures.

Other Construction Types

Most U.S. and foreign nearshore breakwaters built for shore protection have been rubble-mound structures. Rubble-mound construction of nearshore breakwaters is advantageous because of the ability for rubble mounds to dissipate wave energy effectively and provide low reflection coefficients. An extensive amount of research has been conducted for rubble-mound structures,

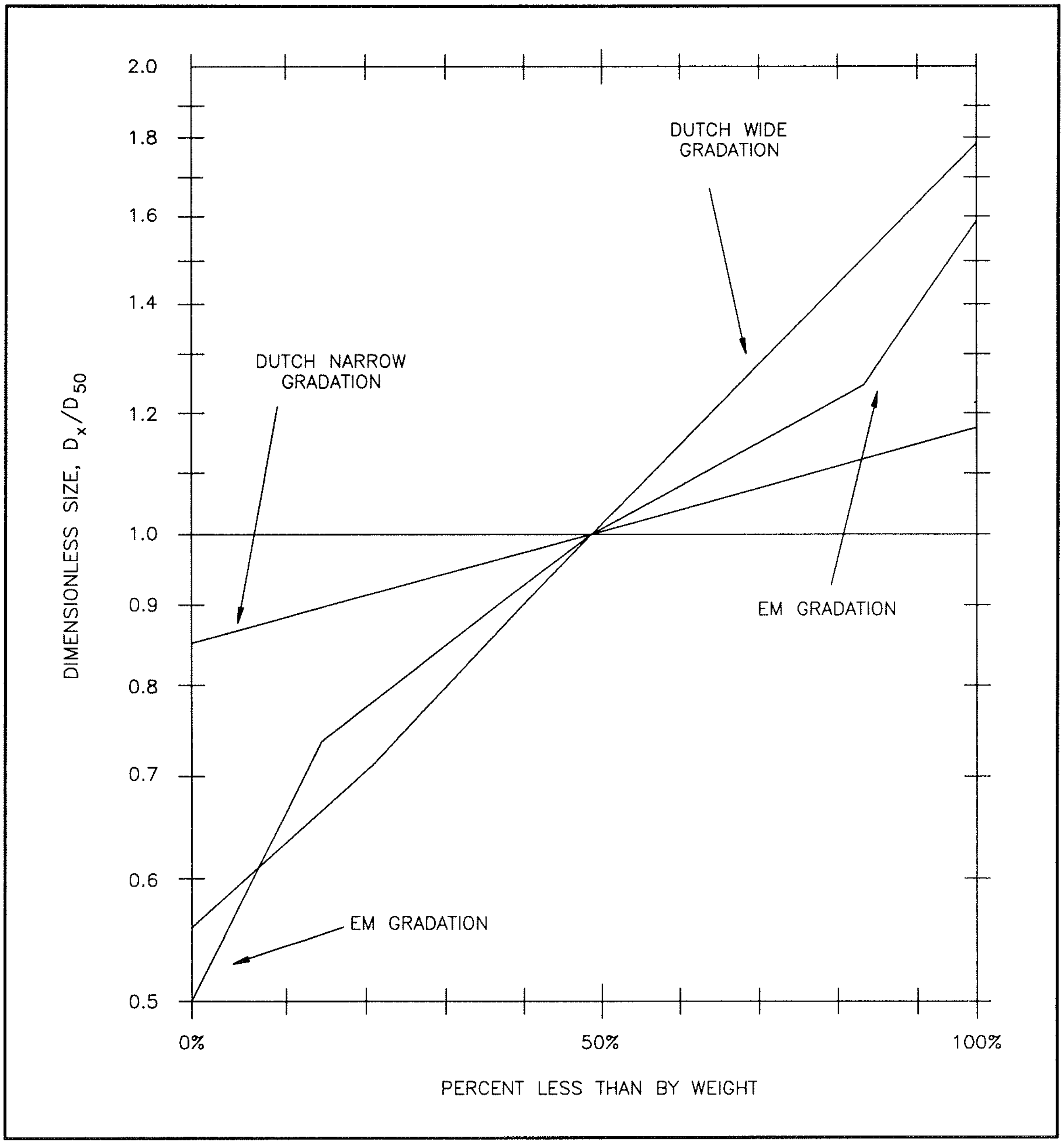


Figure 58. Armor-stone characteristics of Dutch wide gradation, Dutch narrow gradation, and Ahrens (1975) SPM gradation

which provides the designer with confidence in determining the structural stability and resulting performance characteristics of a given structure.

There are numerous proprietary beach erosion control or stabilization systems that function similarly to breakwaters, but are of unique geometry or construction. Most of these systems are precast concrete units, concrete blocks, or flexible structures such as large sand-filled bags placed in various configurations nearshore or in shallow water. Most have undergone limited laboratory testing and many have never been field tested. Generally, these alternative products are designed to function similar to a breakwater. Some have had limited success, and some have not. Some may be applicable in one region and are not valid in another region. However, proponents of various alternative schemes can make unsubstantiated claims of product success. An engineering assessment of the product relative to a specific site is critical prior to its purchase and use.

Some of the structures were evaluated under the Shoreline Erosion Control Demonstration Act, and their performance has been summarized by the Chief of Engineers in his report to Congress (Dunham 1982). All field tests conducted under this program were in sheltered waters and not on the exposed ocean coast. Experience with beach stabilization systems on the open coast is limited, with many cases being selectively reported according to their limited success.

Evaluations of alternative beach stabilization structures should be based on their functional performance, economics relative to traditional types of breakwater construction, aesthetics, and ability to be removed or modified if they do not function as expected or become aesthetically unacceptable (EM 1110-2-1617). The economics and aesthetics of alternative systems often make such systems favorable; however, the lack of laboratory or prototype experience with many alternative structures means limited data are available to help determine the structure's stability and performance characteristics under given design conditions.

Stability of alternative structures is typically not as great a concern as the performance characteristics provided by the structure. Such structures are normally placed in low to moderate wave energy environments where wave loadings are not very severe and structures can often be overdesigned to provide greater stability for a relatively low increase in cost. However, the uncertainty of performance characteristics and their resulting effect on the expected shoreline planform is critical when evaluating alternative structures. Wave transmission characteristics for any structure are critical in determining resulting shoreline configuration as discussed in Chapter 2. Reflection characteristics must be considered for potential scour and navigation problems. Highly reflective near-vertical-faced structures such as sheet-pile breakwaters should be avoided, since extensive toe protection will be required to avoid scour. Also, such structures pose threats to navigation and nearby shorelines due to increased wave activity.

Conservation of energy principles can be applied to initially evaluate the suitability of any structure in terms of functional performance. The basic principle states that all incident wave energy can be accounted for by the summation of energy transmitted, reflected, and dissipated within the structure. For example, a high non-overtopped steel sheet-pile wall can prevent the majority of incident wave energy from being transmitted. Since energy dissipation of the structure is expected to be minimal, the majority of incident energy will be reflected and may potentially cause scour. If the objective is to provide low wave energy transmission and minimal reflective

Inspections

Following construction, periodic inspections of the project should be conducted. Inspections should focus on structural deterioration that affects the functionality of the breakwater. Repairs should be made in a timely manner to prevent further unravelling of the structure. Inspections should also identify potentially hazardous conditions to public safety that may have developed as a result of the structure.

Operations and maintenance manual for local sponsors

EM 1110-2-1617 provides a description of the requirements and guidance for post-construction activities for a shore protection project. Specific performance requirements and guidance for accomplishing the satisfactory maintenance and operation of shore protection works, including coastal structures and beach fill projects, are provided in Engineer Regulation 1110-2-2902. This regulation prescribes operations, maintenance, inspection, and record-keeping procedures required to obtain the intended purposes of shore protection projects.

Post-Construction Monitoring

A post-construction monitoring program to evaluate the functional and structural performance of a detached breakwater project is recommended and described in EM 1110-2-1617. Project monitoring will assist in both the specific project's performance and with developing guidance and methodology for future projects. A monitoring program will allow the identification of specific deficiencies in the performance of a project for which modifications may be made to better meet the project's objectives, and establish if a given structure has sustained damage that may affect its functional capacity. The coastal zone is a complex area; frequent storms can occur and coastal processes can fluctuate over time. From the research standpoint, monitoring of prototype projects, whether successful or not, will provide the data desperately needed to improve design guidance.

A monitoring program should be designed based on the site-specific project. The program must consider not only data collection, but analysis methods and associated costs once the data are obtained. There are several types of basic data that are often included in a monitoring program (EM 1110-2-1617).

Photographic documentation

Photography can provide both qualitative and quantitative information on a breakwater's performance. Controlled, vertical aerial photography can provide quantitative data on ground elevation, shoreline and berm location, offshore shoals, structure geometry, structural deterioration, and beach use changes. Typical aerial photographs for coastal project monitoring are taken at a scale of 1:4800 (1 in. = 400 ft), with a 60-percent overlap for stereographic analysis. Larger scale photography is usually used to examine changes in the elevation of structural components, such as armor units, between successive flights. For structural monitoring, it is important to obtain photography immediately after construction in order to provide a base condition for comparison. The frequency of aerial flights depends on the objectives of the monitoring program. Detailed project monitoring may require quarterly flights, whereas routine inspections may only need annual photography. A more inexpensive, but strictly qualitative method is to obtain periodic ground-level photography to document changes over time in a particular location.

Beach profiles and bathymetric data

Periodic beach profiles can be used to document the accretion, erosion, or stability of the project's shoreline. Beach profile data can assist with both routine evaluation of the project and documentation of storm damages or damages prevented as a result of the project. As with photography, the frequency of profiles depends on the objectives of the monitoring program. Beach changes can occur rapidly after initial construction and may be required more frequently. It is recommended that at least quarterly profiles be conducted to document beach planform development prior to reaching equilibrium. If bathymetric changes due to project construction or seasonal offshore profile changes are required, profile lines will have to extend offshore beyond wading depth. Subaqueous surveys can significantly increase the cost of the monitoring. Additionally, it is important to ensure that for each profile line, the beach and bathymetric data meet and are not vertically offset; otherwise, significant error can be introduced into the analysis.

Spacing of the beach profile lines also depends on the monitoring objectives. General shoreline trends can be documented with distantly spaced profiles, whereas volumetric analysis of erosion and/or accretion requires more closely spaced lines. EM 1110-2-1617 recommends at least three profile lines in the lee of a detached breakwater depending on the structure's length, distance offshore, and other parameters (Figure 62).

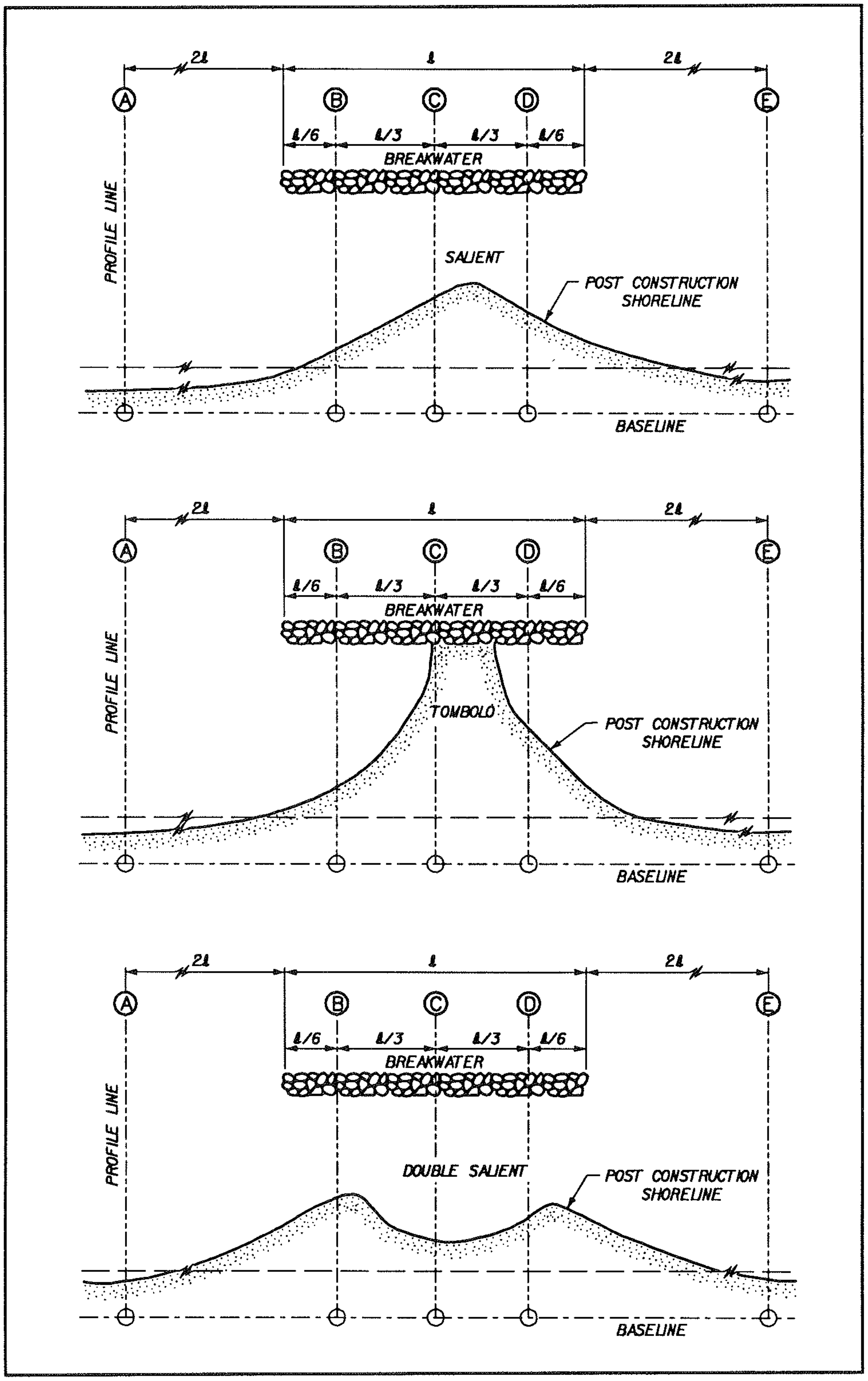


Figure 62. Spacing of profile lines in the lee of a detached breakwater (from EM 1110-2-1617)

Wave conditions

Waves and longshore currents are the driving forces behind beach planform development. Some detailed research monitoring programs may want to examine the cause-and-effect relationships between waves, sediment transport, and a project's functional performance. Data can be obtained at the site using a wave gauge system designed to meet the specific objectives of the study. Generally, wave direction is important when examining functional performance of a project.

Wave data are sometimes collected to evaluate the structural performance of a rubble-mound breakwater. Larger wave heights associated with storms are of primary interest; wave direction is usually of secondary importance.

6 Summary and Conclusions

Report Summary

From prototype experiences, detached breakwaters have proven to be a viable method of shoreline stabilization in the United States. Breakwaters can be designed to retard erosion of an existing beach, promote sedimentation at the lee of the structure to form a new beach, retain placed beach fill material and reduce renourishment intervals, and/or maintain a wide beach for damage reduction and recreation. Low-crested breakwaters can also be combined with dredge material disposal and marsh grass plantings to establish wetlands and control erosion along estuarine shorelines. Most recent prototype applications of detached breakwaters have been along sediment-starved shorelines with low to moderate wave energy such as in the Chesapeake Bay, Great Lakes, and some areas of the Gulf Coast.

This report summarizes and presents design knowledge for both the functional and structural design of detached breakwaters for shoreline stabilization. Functional design of breakwaters in the United States relies on a significant amount of engineering judgement, data from a few existing breakwater projects for comparison, and an understanding of basic coastal processes. The design process is an iterative one. Design guidance used to predict beach response to breakwaters is also presented in Dally and Pope (1986), Pope and Dean (1986), Rosati (1990), and Engineer Manual 1110-2-1617. Guidance on the use of low-crested rubble-mound breakwaters for wetland development purposes is limited, and has been mostly based on experience from a few prototype sites. Ongoing research at WES under the Wetlands Research Program is further investigating and evaluating the use of breakwaters for these purposes.

Functional design techniques and evaluation tools for detached breakwaters can be classified into three categories: empirical relationships, physical and numerical models, and prototype assessment. A three-phase design process is suggested using these tools. First, a desktop study should be conducted employing various empirical relationships to relate proposed structural and site parameters to shoreline response and identify design alternatives. Second, a physical or numerical model study can be used to evaluate beach response to the breakwater project, and to assess and refine alternatives. Finally, if feasible, a prototype test can be conducted to verify and adjust the preliminary design.

Structural design guidance for detached breakwaters involves assessment of structural stability and anticipated performance characteristics for critical and average wave and water level conditions. The use of low-crested breakwaters for beach and wetland stabilization projects has increased since they can be more cost-effective than conventional multilayered navigation breakwaters. Recent guidance to assess structural stability and performance characteristics of low-crested breakwaters is presented in this report.

Additional Research Needs

Continued research relative to detached breakwaters should explore improved techniques to predict beach response and methods to optimize breakwater design. Primary reasons for the limited use of detached breakwaters in the United States are the lack of functional design guidance and high construction costs. Further development of comprehensive criteria is needed for breakwater design in the feasibility, continuing authority, and reconnaissance phases. Current techniques to predict shoreline response and downdrift impacts as a function of structural and site parameters can be insufficient, costly, time-consuming, and not readily available to the designer. Continuing efforts at CERC are completing the development of functional design criteria, in the form of nomographs, based on site and structural parameters (Rosati, Gravens, and Chasten 1992). Additional research is also needed in predicting wave transmission characteristics of detached breakwaters and the resulting influence of transmitted wave energy on beach planform and wetland development. Continued research addressing breakwaters as beach fill stabilization and wetland development structures would be beneficial. Increased benefits from the use of breakwaters in these manners may help justify their costs of construction and encourage breakwater applications in more areas of the United States.

References

Ahrens, J. P. (1975). "Large wave tank tests of riprap stability," CERC TM-51, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

______. (1984). "Reef type breakwaters." *Proceedings, 19th International Conference on Coastal Engineering*, American Society of Civil Engineers, Houston, TX.

______. (1987). "Characteristics of reef breakwaters," Technical Report 87-17, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

______. (1989). "Stability of reef breakwaters," *Journal of Waterway, Port, Coastal and Ocean Engineering*, American Society of Civil Engineering, 221-234.

Ahrens, J. P., and Cox, J. (1990). "Design and performance of reef breakwaters," *Journal of Coastal Research* 61-75.

Bender, T. (1992). "An overview of offshore/headland breakwater projects constructed by the Buffalo District." *Proceedings, Coastal Engineering Practice '92*. American Society of Civil Engineers, Long Beach, CA.

Berenguer, J., and Enriquez, J. (1988). "Design of pocket beaches: The Spanish case." *Proceedings, 21st International Conference on Coastal Engineering*. Malaga, Spain, 1411-25.

Bishop, C. T. (1982). "A review of shore protection by headland control," No. 352, Environment Canada, Burlington, Ontario.

Bottin, R. R., Jr. (1982). "Lakeview Park beach erosion study, Ohio," Letter Report, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Bottin, R. R., Jr., and Chatham, C. E., Jr. (1975). "Design for wave protection, flood control, and prevention of shoaling, Cattaraugus Creek Harbor, New York; hydraulic model investigation," Technical Report

H-75-18, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Bruun, P., ed. (1985). *Design and construction of mounds for breakwaters and coastal protection*. Elsevier Scientific Publishing Co., Amsterdam.

Carver, R. D., and Wright, B. J. (1992). "Use of site specific data for general breakwater design," Technical Report CERC-92-2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Chew, S. Y., Wong, P. P., and Chin, K. K. (1974). "Beach development between headland breakwaters." *Proceedings, 14th International Conference on Coastal Engineering*. Copenhagen, Denmark.

Chu, Y., and Martin, T. (1992). "Beach response to the Redington Shores, Florida breakwater," Miscellaneous Paper CERC 92-8, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Cialone, M. A., Mark, D. J., Chou, L. W., Leenknecht, D. A., Davis, J. E., Lillycrop, L. S., and Jensen, R. E. (1992). "Coastal Modeling System (CMS) user's manual," Instruction Report CERC-91-1, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Coastal Engineering Research Center. (1984). "Empirical methods for the functional design of detached breakwaters for shoreline stabilization," CETN-III-43, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Coleman, W. E., Jr. (1992). "Chesapeake Bay field modeling and monitoring projects." *Proceedings, Coastal Engineering Practice '92*. American Society of Civil Engineers, Long Beach, CA.

Curren, C. R., and Chatham, C. E., Jr. (1977). "Imperial Beach, California, design of structures for beach erosion control," Technical Report H-77-15, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

_______. (1980). "Oceanside Harbor and Beach, California design of structures for harbor improvement and beach erosion control," Technical Report H-80-10, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Daemen, I. F. R. (1991). "Wave transmission at low-crested breakwaters," M.S. thesis, Delft University of Technology, Faculty of Civil Engineering, Delft.

Dally, W. R., and Pope, J. (1986). "Detached breakwaters for shore protection," Technical Report CERC-86-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Dunham, J. W. (1982). "Chief of Engineers' final report to Congress on the Shoreline Erosion Control Demonstration Act," U.S. Army Corps of Engineers, Office of the Chief of Engineers, Washington, D.C.

Ebersole, B. A. (1982). "Atlantic coast water level climates," WIS Report 7, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Eckert, J. W., and Callender, G. (1987). "Geotechnical engineering in the coastal zone," Instruction Report CERC-87-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Fulford, E. T. (1985). "Reef type breakwaters for shoreline stabilization." *Proceedings, Coastal Zone '85*. American Society of Civil Engineers.

Fulford, E. T., and Usab, K. M. (1992). "Bay Ridge, Ann Arundel County, Maryland, offshore breakwater and beach fill design." *Proceedings, Coastal Engineering Practice '92*. American Society of Civil Engineers, Long Beach, CA.

Goldsmith, V. (1990). "Engineering performance of detached breakwaters along the coast of Israel," draft report prepared for the U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Hunter College, New York.

Gourlay, M. R. (1981). "Beach processes in the vicinity of offshore breakwaters." *Proceedings, Fifth Australian Conference on Coastal and Ocean Engineering*, Perth, Australia, 129-34.

Gravens, M. B. (1990). "Computer program: GENESIS Version 2.0," Coastal Engineering Technical Note, CETN-II-21, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

______. (1991). "Development of an input data set for shoreline change modeling." *Proceedings, Coastal Sediments '91*. American Society of Civil Engineers, 1800 -13.

______. (1992). "User's guide to the shoreline modeling system," Instruction Report CERC-92-1, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Gravens, M. B., and Scott, S. (1993). "Analysis of wave data for use in sediment transport calculations." *Proceedings, 6th Annual Beach Preservation Technology*. Florida Shore and Beach Preservation Association.

Gravens, M. B., Kraus, N. C., and Hanson, H. (1991). "GENESIS: Generalized model for simulating shoreline change; Report 2, workbook and system user's manual," Technical Report CERC-89-19, U.S. Army

Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Hallermeier, R. J. (1983). "Sand transport limits in coastal structure design." *Proceedings, Coastal Structures '83.* March 9-11, Arlington, Virginia, American Society of Civil Engineers, 703-16.

Hanson, H., and Kraus, N. C. (1989a). "GENESIS - a generalized shoreline change numerical model," *Journal of Coastal Research* 5, 1-27.

______. (1989b). "GENESIS: Generalized model for simulating shoreline change, Report 1, technical reference," Technical Report CERC-89-19, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

______. (1990). "Shoreline response to a single transmissive detached breakwater." *Proceedings, 22nd International Conference on Coastal Engineering.* Delft, The Netherlands, 2034-46.

______. (1991). "Numerical simulation of shoreline change at Lorain, Ohio," *Journal of Waterway, Port, Coastal, and Ocean Engineering* 117, (1 January/February) 1-18.

Hanson, H., Kraus, N. C., and Nakashima, L. D. (1989). "Shoreline change behind transmissive detached breakwaters." *Proceedings, Coastal Zone '89.* American Society of Civil Engineers, Charleston, SC.

Hardaway, C. S., Jr., and Gunn, J. R. (1991a). "Working breakwaters," *Civil Engineering Magazine*, American Society of Civil Engineers.

______. (1991b). "Headland breakwaters in Chesapeake Bay." *Proceedings, Coastal Zone '91.* American Society of Civil Engineers, Long Beach, CA, 1267-81.

Hardaway, C. S., Jr., Gunn, J. R., and Reynolds, R. N. (1993). "Breakwater design in the Chesapeake Bay: dealing with the end effects." *Proceedings, Coastal Zone '93, Coastal Engineering Considerations in Coastal Zone Management.* American Society of Civil Engineers, New Orleans, LA, 27-41.

Harris, D. L. (1981). "Tides and tidal datums in the United States," Special Report 7, Coastal Engineering Research Center, Fort Belvoir, VA.

Harris, M. M., and Herbich, J. B. (1986). "Effects of breakwater spacing on sand entrapment," *Journal of Hydraulic Research* 24 (5), 347-57.

Hubertz, J. M., Brooks, R. M., Brandon, W. A., and Tracy, B. A. (1993). "Hindcast wave information for the U.S. Atlantic Coast," WIS Report 30, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hudson, R. Y., Herrmann, F. A., Jr., Sager, R. A., Whalin, R. W., Keulegan, G. G., Chatham, C. E., Jr., and Hales, L. Z. (1979). "Coastal Hydraulics Models," Special Report No. 5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hughes, S. A. *Physical models and laboratory techniques in coastal engineering*. In preparation, World Scientific Publishing Co., Singapore.

Inman, L. D., and Frautschy, J. D. (1966). "Littoral processes and the development of shorelines." *Proceedings, Coastal Engineering*. Santa Barbara, CA, 511-36.

Japanese Ministry of Construction. (1986). *Handbook of offshore breakwater design*. River Bureau of the Ministry of Construction, Japanese Government, translated from Japanese for the Coastal Engineering Research Center.

Jensen, R. E. (1983). "Atlantic coast hindcast, shallow water, significant wave information," WIS Report 9, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Jensen, R. E., Hubertz, J. M., Thompson, E. F., Reinhard, R. D., Borup, B. J., Brandon, W. A., Payne, J. B., Brooks, R. M., and McAneny, D. S. (1992). "Southern California hindcast wave information," WIS Report 20, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Kamphuis, J. W. (1975). "Coastal mobile bed model--does it work?" *Proceedings of the Second Annual Symposium on Modeling Techniques*. 993-1009.

Keulegan, G. H. (1966). "Model laws for coastal and estuarine models." *Estuary and coastline hydrodynamics*. A.T. Ippen, ed., McGraw-Hill, New York.

Knutson, P. L., Allen, H. H., and Webb, J. W. (1990). "Guidelines for vegetative erosion control on wave-impacted coastal dredged material sites," Technical Report D-90-13, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Kraft, K., and Herbich, J. B. (1989). "Literature review and evaluation of offshore detached breakwaters," Texas A&M University, Report No. COE-297, prepared for the U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Kraus, N. C. (1983). "Applications of a shoreline prediction model." *Proceedings, Coastal Structures '83*. American Society of Civil Engineers, 632-45.

Kraus, N. C. (Editor). (1990). "Shoreline change and storm-induced beach erosion modeling: A collection of seven papers," Miscellaneous Paper CERC-90-2, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Landin, M. C., Webb, J. W., and Knutson, P. L. (1989). "Long-term monitoring of eleven Corps of Engineers habitat development field sites built by dredged material, 1974-1987," Technical Report D-89-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Langhaar, H. L. (1951). *Dimensional analysis and theory of models*. Wiley, New York.

Larson, M., and Kraus, N. C. (1989). "SBEACH: Numerical model for simulating storm-induced beach change; Report 1: Empirical foundation and model development," Technical Report CERC-89-9, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Larson, M., Hanson, H., and Kraus, N. C. (1987). "Analytical solutions of the one-line model of shoreline change," Technical Report CERC-87-15, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Larson, M., Kraus, N. C., and Byrnes, M. R. (1990). "SBEACH: Numerical model for simulating storm-induced beach change; Report 2: Numerical formulation and model tests," Technical Report CERC-89-9, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Leenknecht, D. A., Szuwalski, A., and Sherlock, A. R. (1993). "Automated Coastal Engineering System: User's Guide," Version 1.07, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Le Mehaute, B. (1970). "A comparison of fluvial and coastal similitude." *Proceedings, 12th Conference on Coastal Engineering*. American Society of Civil Engineers, Vol. 2, 1077-96.

Lesnik, J. (1979). "An annotated bibliography on detached breakwaters and artificial headlands," Miscellaneous Report CERC-79-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Maquet, J. (1984). "Construction methods and planning." *Breakwaters - design and construction*. Institute of Civil Engineers, London.

Moffatt and Nichol, Engineers. (1983). "Construction materials for coastal structures," Special Report 10, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Mohr, M. C., and Ippolito, M. (1991). "Initial shoreline response at the Presque Isle erosion control project." *Proceedings, Coastal Sediments '91.* American Society of Civil Engineers, Seattle, WA.

Nakashima, L. D., Pope, J., Mossa, J., and Dean, J. L. (1987). "Initial response of a segmented breakwater system, Holly Beach, Louisiana." *Proceedings, Coastal Sediments '87.* 1399-1414.

Nir, Y. (1982). "Offshore artificial structures and their influence on the Israel and Sinai Mediterranean beaches." *Proceedings, 18th International Conference on Coastal Engineering.* American Society of Civil Engineers, 1837-56.

Noble, R. M. (1978). "Coastal structures' effects on shorelines." *Proceedings, 17th International Conference on Coastal Engineering.* Sydney, Australia, American Society of Civil Engineers, 2069-85.

Noda, E. K. (1971). "Coastal moveable-bed scale-model relationship," TETRET-P-71-191-7, Tetra Tech, Inc., Pasadena, CA.

Noda, H. (1984). "Depositional effects of offshore breakwater due to onshore-offshore sediment movement." *Proceedings, 19th International Conference on Coastal Engineering.* American Society of Civil Engineers, Houston, TX, 2009-25.

Perlin, M., and Dean, R. G. (1983). "A numerical model to simulate sediment transport in the vicinity of coastal structures," Miscellaneous Report MR-83-10, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Pope, J. (1989). "Role of breakwaters in beach erosion control." *Proceedings, Beach Preservation Technology '89.* Tampa, Florida, Florida Shore and Beach Preservation Association, 167-76.

Pope, J., and Dean, J. L. (1986). "Development of design criteria for segmented breakwaters." *Proceedings, 20th International Conference on Coastal Engineering.* Taipei, Taiwan, American Society of Civil Engineers, 2144-58.

Rogers, S. M. (1989). "Erosion control: Marsh and low-cost breakwater." *Proceedings, Coastal Zone '89.* American Society of Civil Engineers, Charleston, SC.

Rosati, J. D. (1990). "Functional design of breakwaters for shore protection: Empirical methods," Technical Report CERC-90-15, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Rosati, J. D., and Truitt, C. L. (1990). "An alternative design approach for detached breakwater projects," Miscellaneous Paper CERC-90-7, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Rosati, J. D., Gravens, M. B., and Chasten, M. A. (1992). "Development of detached breakwater design criteria using a shoreline response model." *Proceedings, Coastal Engineering Practice '92*. American Society of Civil Engineers, Long Beach, CA.

Rosati, J. D., Wise, R.A., Kraus, N.C., and Larson, M. (1993). "SBEACH: Numerical model for simulating storm-induced beach change; Report 3, user's manual," Instruction Report CERC-93-2, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Rosen, D. S., and Vadja, M. (1982). "Sedimentological influence of detached breakwaters." *Proceedings, 18th International Conference on Coastal Engineering*. American Society of Civil Engineers, 1930-1949.

Scheffner, N. W. (1988). "N-line (bathymetric changes) model," Coastal Engineering Technical Note CETN-II-10, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Scheffner, N. W., and Rosati, J. D. (1987). "A user's guide to the n-line model: A numerical model to simulate sediment transport in the vicinity of coastal structures," Instruction Report CERC-87-4, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Schuring, D. J. (1977). *Scale Models in Engineering; Fundamentals and Applications*. Pergamon Press, New York.

Seabergh, W. C. (1983). "Design for prevention of beach at Presque Isle Beaches, Erie, Pennsylvania," Technical Report HL-83-15, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Seelig, N. (1980). "Two-dimensional tests of wave transmission and reflection characteristics of laboratory breakwaters," Technical Report 80-1, U.S. Army Corps of Engineers, Fort Belvoir, VA.

Seelig, W. N., and Walton, T. L., Jr. (1980). "Estimation of flow through offshore breakwater gaps generated by wave overtopping," Coastal Engineering Technical Aid (CETA) 80-8, Coastal Engineering Research Center, Fort Belvoir, VA.

Seiji, M., Uda, T., and Tanaka, S. (1987). "Statistical study on the effect and stability of detached breakwaters," *Coastal Engineering in Japan* 30(1), 131-41.

Sheppard, D.M., and Hearn, J.K. (1989). "Performance and stability of low-crested breakwaters," Coastal and Oceanographic Engineering Department, University of Florida, Gainesville, FL.

Shore Protection Manual. (1984). 4th ed., 2 Vols. U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, U.S. Government Printing Office, Washington, DC.

Silvester, R. (1976). "Headland defense of coasts." *Proceedings, 15th International Conference on Coastal Engineering*. American Society of Civil Engineers, Honolulu, HI, 1394-1406.

Silvester, R., and Hsu, J. R. C. (1993). *Coastal stabilization, innovative concepts*. Prentice-Hall, Inc., Englewood Cliffs, NJ.

Silvester, R., Tsuchiya, Y., and Shibano. Y. (1980). "Zeta Bays, pocket beaches and headland control." *Proceedings, 17th International Conference on Coastal Engineering*. American Society of Civil Engineers, Sydney, Australia.

Smith, O. P. (1986). "Cost-effective optimization of rubble-mound breakwater cross sections," Technical Report CERC-86-2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Sonu, C. J., and Warwar, J. F. (1987). "Evolution of sediment budgets in the lee of a detached breakwater." *Proceedings, Coastal Sediments '87.* American Society of Civil Engineers, New Orleans, LA, 1361-68.

Suh, K., and Dalrymple, R. A. (1987). "Offshore breakwaters in laboratory and field," *Journal, Waterway, Port, Coastal, and Ocean Engineering* 113 (2), 105-21.

Toyoshima, O. (1972). *Coastal engineering for practicing engineers-beach erosion,* Morikita Publishing Co., Tokyo, Japan, 227-317. (English translation of Chapter 8, "Offshore breakwaters," available through the Coastal Engineering Research Center, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.)

Toyoshima, O. (1974). "Design of a detached breakwater system." *Proceedings, 14th International Conference on Coastal Engineering*. American Society of Civil Engineers, Copenhagen, Denmark, 1419-31.

______. (1976). "Changes of sea bed due to detached breakwaters." *Proceedings, 15th International Conference on Coastal Engineering*. American Society of Civil Engineers, Honolulu, HI, 1572-89.

______. (1982). "Variation of foreshore due to detached breakwaters." *Proceedings, 18th International Conference on Coastal Engineering*. American Society of Civil Engineers, 1873-92.

U.S. Army Corps of Engineers (1958). "Bearing capacity of soils," Engineer Manual 1110-2-1903, Washington, DC.

______ (1986). "Design of breakwaters and jetties," Engineer Manual 1110-2-2904, Washington, DC.

______ (1986). "Storm surge analysis and design water level determination," Engineer Manual 1110-2-1412, Washington, DC.

______ (1987). "Beneficial uses of dredged material," Engineer Manual 1110-2-5026, Washington, DC.

______ (1989). "Environmental engineering for coastal protection," Engineer Manual 1110-2-1204, Washington, DC.

______ (1989). "Water levels and wave heights for coastal engineering design," Engineer Manual 1110-2-1414, Washington, DC.

______ (1989). "Prescribed procedures for the maintenance and operation of shore protection works, ER 1110-2-2902, Washington, DC.

______ (1991). "Design of pile structures and foundations," Engineer Manual 1110-2-2906, Washington, DC.

______ (1992). "Coastal groins and nearshore breakwaters," Engineer Manual 1110-2-1617, Washington, DC.

______ (1992). "Coastal littoral transport," Engineer Manual 1110-2-1502, Washington, DC.

U.S. Army Engineer District, Buffalo. (1980). "Presque Isle Peninsula, Erie, Pennsylvania," Final Phase I, General Design Memorandum including Environmental Impact Statement, Main Report and Appendices, Volumes I and II, Buffalo, NY.

U.S. Army Engineer District, Buffalo (1986). "Sims Park, Euclid, Ohio, Detailed Project Report on Shoreline Erosion/Beach Restoration on Lake Erie," Buffalo, NY.

U.S. Army Engineer District, Detroit. (1986). "Monthly bulletin of lake levels for the Great Lakes," Detroit, MI.

U.S. Army Engineer District, Jacksonville. (1984). "Pinellas County, Florida beach erosion control project, Sand Key segment," Jacksonville, FL.

U.S. Department of Agriculture (1992). "Wetland restoration, enhancement, or creation." Chapter 13, *National engineering field handbook.* Soil Conservation Service, U.S. Department of Agriculture, Washington, DC.

Van der Meer, J. W. (1987). "Stability of breakwater armour layers - design formulae." Coastal Engineering 11, 219-39.

______. (1988). "Rock slopes and gravel beaches under wave attack," Ph.D. Thesis, Delft University of Technology, Delft, The Netherlands. (Also Delft Hydraulics Publication No. 396.)

______. (1990). "Low-crested and reef breakwaters," Delft Hydraulics Report H 986, Delft, The Netherlands.

______. (1991). "Stability and transmission at low-crested structures." Delft Hydraulics Publication No. 453, Delft, The Netherlands.

Van der Meer, J. W., and Pilarczyk, K. W. (1987). "Stability of breakwater armor layers, deterministic and probabilistic designs," Delft Hydraulics Publication No. 378, Delft, The Netherlands.

Walker, J. R., Clark, D., and Pope, J. (1980). "A detached breakwater system for shore protection." *Proceedings, 17th International Conference on Coastal Engineering*. American Society of Civil Engineers, Sydney, Australia, 1968-87.

Yalin, S. (1971). *Theory of hydraulic models*. MacMillan, London.

Appendix A
Case Design Example of a Detached Breakwater Project

Introduction

This appendix was prepared by Mr. Edward T. Fulford of Andrews Miller and Associates, Inc. The community of Bay Ridge, Anne Arundel County, Maryland, is located on the western shore of the Chesapeake Bay near Annapolis as shown in Figure A1. The shoreline is about 2,250 ft in length and is composed of a sandy beach fronting a bank ranging in height from about 8 to 24 ft. Bay Drive runs parallel to the shoreline in this area and a sewer line also parallels the shoreline along the western side of Bay Drive.

As a result of continued erosion of the bank and shoreline at a rate of 2 to 3 ft per year, a feasibility study was completed in January 1987 which recommended the construction of offshore breakwaters and beach fill as the only effective alternative to provide erosion control and storm protection for the area without eliminating the existing recreational use of the beach. Figure A2 shows the eroded condition of the beach prior to project construction.

In September 1990, detailed design of the project was completed and construction was initiated in November 1990. The following paragraphs discuss the design and construction of the offshore breakwater and beach fill project and the preliminary post-construction performance of the project.

Coastal Processes

Winds

The wind conditions at Bay Ridge were developed from wind observations at the Baltimore-Washington International Airport. The length of the wind

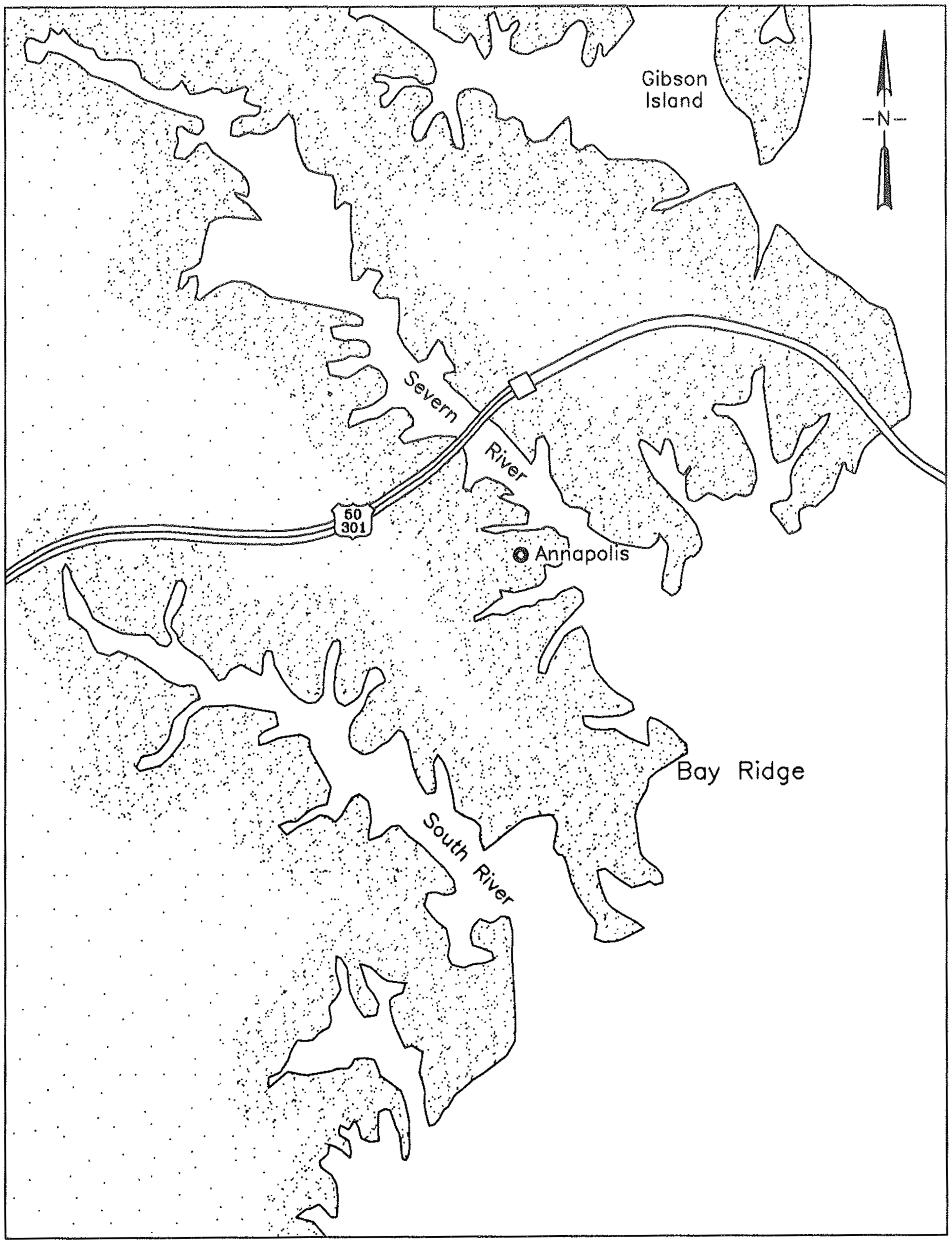


Figure A1. Location Map

record used in this analysis was 30 years. Observed wind data were adjusted by appropriate conversions to overwater conditions and an elevation of 10 m. Statistical analyses were then performed to determine return intervals for wind conditions.

The approach used to estimate the return intervals for winds was to divide the wind observations into the 16 principal compass directions. The probability of observing a particular wind condition is the product of the probability of observing a particular wind speed and the probability of observing a particular wind direction. In order to determine values that

Figure A2. Existing shoreline condition

properly correspond to this product, all observations were categorized according to speed and direction, forming a probability matrix. The matrix, when contoured, exhibited various combinations of wind speed and direction that correspond to each probability of occurrence. The matrix was adjusted to account for the length of record from the measurement site. For wave and water level estimates, the combination of wind speed and direction was chosen that would potentially generate the highest waves and winds. For Bay Ridge, the following design wind conditions were determined:

Table A1
Design Wind Conditions

Event	Wind Speed (knots)	Wind Direction
1-year	32	SSE
10-year	43	SSE
25-year	49	SSE
50-year	65	SSE

Water levels

The wind speed and directions corresponding to each design return interval were uniformly applied to a simple numerical finite difference storm surge model of the Chesapeake Bay. The model used was a simplified model originally developed by Reid and Bodine (1968) for the Texas coast. The model calculated the wind-induced setup of the water level throughout the Bay.

The storm surge model used a time-stepping finite difference numerical algorithm that solved appropriate differential equations representing a flow system. The equations and momentum and continuity expressions contain terms that simulate the following processes:

- Coriolis forces
- Surface wind stresses
- Bottom stresses
- Advection
- Surface slope currents

Land boundaries in the simple model were treated as vertical walls; however, the model did not have the capability to simulate inland flooding.

Based upon application of the simple storm surge model representation of the Chesapeake Bay and Bay Ridge areas, the storm surge levels corresponding to each design return interval for the project area are shown in Table A2.

Table A2
Design Water Levels

Event	Surge Level (ft, mlw)
1-year	3.6
10-year	4.5
25-year	5.2
50-year	6.0

Waves

Wave conditions for the design of shore protection structures at Bay Ridge were generated using an array of numerical models and finite difference grid scales.

Large scale, or "offshore," wave conditions in the Chesapeake Bay were calculated using a time-stepping directional spectral wave model. Directional spectral wave models are generally more accurate than other methods of determining wave conditions on the Chesapeake Bay primarily because the Bay is considered both a narrow and shallow water fetch over which the waves are generated. Simpler techniques for determining wave conditions do not account for land boundaries on the sides of narrow fetches and do not account for the predominance of shoal areas such as those in the Bay. Significant errors in wave estimates during extreme events can result by ignoring these physical constraints on wave generation and propagation. The Chesapeake Bay was initially digitized into a 2.5-nautical mile finite difference grid, over which winds corresponding to each design event were applied.

The offshore wave conditions generated in the initial wave model application were used as input to a finer scale simulation. The simulation was performed using the same directional spectral wave model at a finite difference grid scale of 500 ft including all important nearshore wave transformation processes, including wave refraction, shoaling, wave-wave interactions, bottom friction, etc. The nearshore wave conditions for each approach direction at the -3-ft mean low water (mlw) contour are presented in Table A3 for each design event. The design water depth at that location includes the corresponding storm surge plus a 1-ft astronomical tide.

Table A3
Design Wave Conditions

Event	Wave Height (H_s, ft)	Period (sec)	Direction
1-year	5.0	5.6	SSE
10-year	5.6	6.5	SSE
25-year	6.5	7.6	SSE
50-year	7.2	9.7	SSE

Sediment transport

Longshore transport. Preliminary analyses of aerial photos and wind distributions indicated that the predominant net longshore littoral drift in the project area is small and in a northerly direction. To gain a better insight into this process, several techniques were used.

Energy flux method. This method is based on the assumption that the longshore transport rate of littoral material can be computed from the longshore component of energy flux in the surf zone according to the following equation:

$$Q = 7500\, P_{ls} \qquad \text{(A1, Equation 4-50, SPM)}$$

The longshore energy flux in the surf zone is approximated by assuming conservation of energy flux in shoaling waves, using small-amplitude wave theory, and then evaluating the energy flux relation at the breaker position. This energy flux is then related to sediment transport through an empirical relationship. The procedure used in this type of analysis is to first develop the wave climate for an area, consisting of wave heights, periods, and breaking wave angles between the wave crests and the shoreline and the percent occurrence of these conditions. These wave parameters are then used in the empirical relationship to determine the amount of sediment that could be transported by each wave condition.

For this analysis, each wind speed and direction combination was applied to the wave model grid, yielding a nearshore wave height/period/direction combination resulting from that wind. These wave characteristics were converted to longshore energy flux potential and transport potential and weighted by their individual probability of occurrence. Summing the relative contributions of the wave resulting from each wind speed/direction combination yielded a net longshore sediment transport potential of 13,300 cu yd traveling northerly along the Bay Ridge shoreline.

Aerial photography analysis. Seven sets of aerial photographs of the project area shoreline from 1962 to 1985 were analyzed for evidence of longshore sediment transport direction. The results of the aerial photography analysis and field observations supported the results of the analytical determination of the longshore transport in the area. Overall, there appears to be a net longshore transport to the north along the study area shoreline with some occurrence of southerly transport. The best estimate for the magnitude of the net transport rate is approximately 5,000 to 10,000 cu yd/yr to the north.

Structural Breakwater Design

Design wave and water level

The level of structure design was the 25-year storm event. Based on the numerical modeling analysis, the design wave height H_s, wave period T, and storm surge *DSWL* for this event are:

H_s = 6.5 ft
T = 7.6 sec
DSWL = +5.2 ft mlw

Breakwater stone size and cross section

Selection of the armor stone size to withstand the design wave conditions was based on the stability formula developed at the U.S. Army Engineer Waterways Experiment Station. This formula is as follows:

$$W = \frac{w_r H^3}{K_D \left(S_r - 1\right)^3_{\cot\theta}} \qquad \text{(A2, Equation 7-116, SPM)}$$

where

W = weight in pounds of an individual armor unit in the primary cover layer. The stones comprising the primary cover layer range from about 0.75 W to 1.25 W, with about 50 percent of the individual stones weighing more than W

w_r = unit weight of stone; 165 lb/ft^3

H = design wave height at the structure; 6.5 ft

S_r = specific gravity of the armor unit, relative to the water at the structure ($S_r = w_r/w_w$); 2.58

w_w = unit weight of water; 64.0 lb/ft^3

θ = angle of the structure slope measured from the horizontal in degrees; cot θ = 1.5

K_D = stability coefficient for rough angular armor units; = 2.0

For the design conditions at the site, W = 3,300 lb. The acceptable range for W is 2,500 to 4,500 lb with 50 percent of the individual stones weighing more than 3,300 lb.

The bedding and core stone directly beneath the primary armor units is 3-in. to 8-in. stone.

The crest width of the breakwaters is calculated from the following equation (SPM, 1984):

$$B = n\, k_d\, (W/w_r)^{1/3} \qquad \text{(A3, Equation 7-120, SPM)}$$

where

B = crest width, ft
n = number of stones (n = 3 is recommended minimum)
k_d = layer coefficient; 1.00
w_r = unit weight of stone; 165 lb/ft^3

Using this equation, the crest width is calculated to be 9.5 ft.

Since the breakwaters will be exposed to breaking waves, a quarrystone toe berm is required to support the primary cover layers. The width of this berm is 6 ft and the thickness of the berm is 3 ft. A typical section of the breakwater is shown in Figure A3.

Foundation analysis

Vibracores were taken at four offshore locations beneath the area of the proposed offshore breakwaters. A semi-portable coring system was used. Cores ranging in length from 4 to 5 ft were obtained. Analysis of these cores indicated that a surface sand layer overlies the entire area, ranging in thickness from 10 to 20 in. and that no settlement is expected, either initially or in the long term.

Functional Breakwater Design

Shore-parallel breakwaters constructed offshore provide protection by reducing the amount of wave energy reaching the leeward water and shore

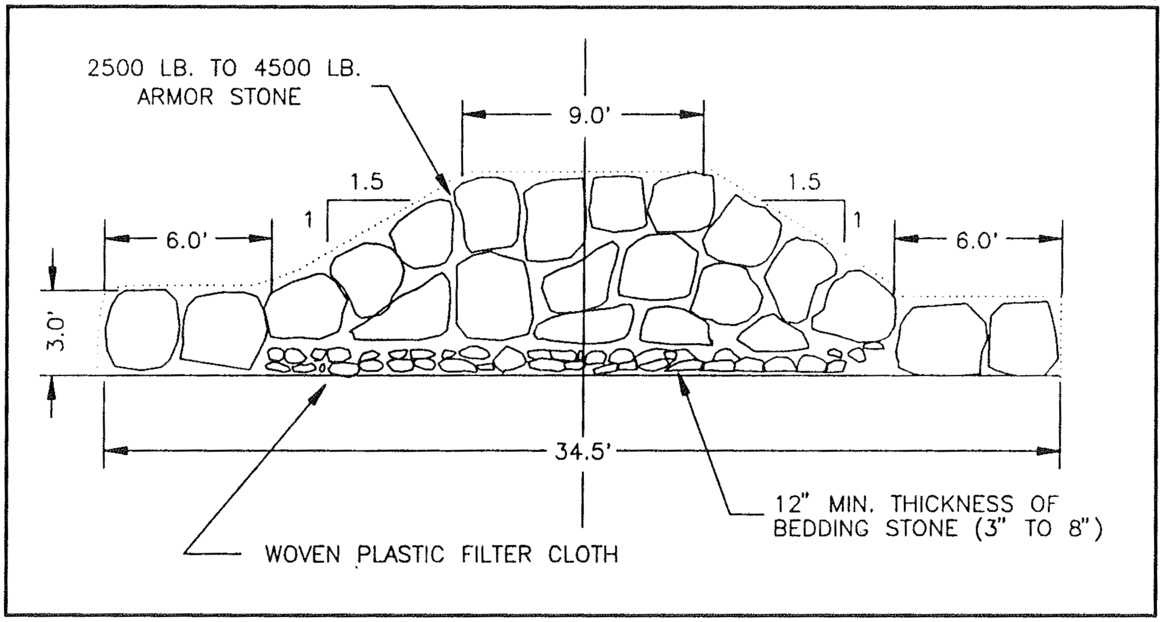


Figure A3. Typical breakwater section

area. As discussed in the *Shore Protection Manual* (1984), the shoreline response resulting from the construction of an offshore breakwater is governed by the resulting changes in the longshore sediment transport and the onshore-offshore sediment transport in the vicinity of the breakwater. For obliquely incident waves, the longshore transport rate in the lee of the structure will initially decrease, causing deposition of some of the longshore drift. A beach salient is formed, which will continue to grow until either the longshore rate past the structure is reestablished or a tombolo (attachment of the salient to the breakwater) is formed.

For the project area, the objective was to reduce the wave energy reaching the eroding shoreline to a level that would not cause erosion during storm events. This objective was to be accomplished without creating any adverse effects along the adjacent shoreline areas. Of the two shoreline responses, salient formation was preferred so that the breakwaters would not become attached to shore creating a barrier to littoral drift (i.e., tombolo formation). Tombolo formation is prevented by allowing sufficient wave energy to enter the protected region.

Breakwater length versus distance offshore

Pope and Dean (1986) investigated seven offshore breakwater projects in the United States and concluded that the beach response in the lee of the breakwaters is a direct result of the amount of wave energy reaching the beach. A classification scheme was developed where the lowest wave energy in the lee of the breakwaters results in tombolo formation and little or no response of the shoreline occurs when high wave energy reaches the shoreline.

The five beach response planforms used in this classification scheme are as follows:

a. PERMANENT TOMBOLOS - Very little wave energy reaches the shore and the beach is stable with little transport along the shore.

b. PERIODIC TOMBOLOS - One or more of the breakwater segments are periodically backed by tombolos with a periodic trapping of littoral material followed by a release of a "slug" of sediment to the downdrift shoreline.

c. WELL-DEVELOPED SALIENTS - These planforms occur when somewhat higher wave energy reaches the lee of the structures and they are characterized by a balanced sediment budget. Longshore moving material enters and leaves the project at approximately the same rate.

d. SUBDUED SALIENTS - In this case, the shoreline response is not as pronounced, and the amplitude of the salient is of lower relief.

e. NO SINUOSITY - High wave energy reaches the beach in this case resulting in little if any shoreline response.

Ahrens and Cox (1990) developed an empirical expression for a beach response index based on the data from the seven offshore breakwater projects presented in Pope and Dean (1986). This index is based on the ratio of the length of the breakwater L_s to the offshore distance of the breakwater X. The values of this index for the five beach response classifications of Pope and Dean (1986) are shown in Table A4.

For the project area, various combinations of breakwater lengths and offshore distances, along with the corresponding beach response index, were evaluated as shown in Table A5.

In order to maximize the protection to the project area shoreline and maintain the longshore transport rate along the shoreline, the desired planform ranged from subdued salients to well-developed salients. To achieve this planform, the combination of a breakwater length of 100 ft and offshore distance of 133 ft was selected.

Breakwater segmentation

A primary area of concern for the project area was the magnitude of diffracted waves in the lee of the gaps. Waves will enter the breakwater gaps and diffract behind the structures and toward the shoreline. Upon reaching the shoreline, sufficient beach width and berm height are required to dissipate this wave energy prior to its reaching the bank toe. If the existing beach width and height are not sufficient to dissipate the wave energy, the options are to design the breakwaters to further decrease the wave energy propagating

Table A4
Beach Response Classifications (from Pope and Dean (1986))

Beach Response Index	Classification
1.0	Permanent tombolos
2.0	Periodic tombolos
3.0	Well-developed salients
4.0	Subdued salients
5.0	No sinuosity

Table A5
Breakwater Length/Distance Offshore vs Beach Response

L_s	X	L_s	X	L_s	X	L_s/X	Beach Response (Ahrens and Cox 1990)
50 50	200 100	75 75	300 150	100 100	400 200	.25 .50	5.0/no sinuosity 4.5/no sinuosity
50	75	75	112	100	150	.67	4.2/subdued salients
50	67	75	100	100	133	.75	4.1/subdued salients
50	50	75	75	100	100	1.00	3.7/subdued salients
50	40	75	60	100	80	1.25	3.3/well-developed salients
50	33	75	50	100	67	1.50	3.0/well-developed salients
50	29	75	43	100	57	1.75	2.7/well-developed salients
50	25	75	38	100	50	2.00	2.5/periodic tombolos

through the gaps (e.g., smaller gaps, with the resulting increase in the length of breakwater segments) or to add beach fill to the shoreline area. In the latter application, the function of the breakwater system is to reduce the wave energy level such that the beach fill will form a stable equilibrium planform and dissipate the remaining wave energy prior to its reaching the toe of the bank.

To evaluate the potential wave transmission characteristics of various breakwater gaps, nearshore diffraction diagrams were developed for the lee of the breakwaters for each design event. Analysis of the diagrams indicated that the 50-yr design wave height of 7.2 ft would be reduced to about 3 ft at a distance of about 45 ft from the toe of the bank for breakwater gap widths of 100 ft, the minimum gap width considered practical for the area. Assuming a breaking wave parameter k equal to 0.78, this wave would break in a depth of water of about 3.8 ft. With the existing beach berm at +2.5 ft mlw and the design storm tide at +6.0 ft mlw, these waves would break directly on the bank toe and cause significant erosion.

In lieu of reducing the diffracted design wave height by narrowing the breakwater gap width, beach fill placement was selected to provide the desired protection for the bank area. The beach fill plan consisted of raising the height of the existing berm to +6.0 ft mlw for a width of 30 ft from the toe of the existing bank and then sloping 1H on 8V to the existing bottom.

With the storm berm in place at a height of +6.0 ft mlw, wave heights near the toe of the bank would be depth-limited to less than 1 ft during the 50-yr storm analyzed for functional performance (at the sponsor's request). Following its placement, the beach fill would be expected to evolve to a stable planform with salients forming behind each breakwater and embayments opposite each gap. As a result of this process, the mean high water line (mhwl) behind the breakwaters would advance bayward and the mhwl opposite the gaps would recede shoreward. Analysis of the diffracted wave patterns in the area and the performance of numerous other offshore breakwater configurations indicate that recession of the mhwl opposite the gaps would be on the order of 15-20 ft.

During the evolution of the shoreline, the slope of the beach fill would be expected to evolve to a more natural and milder slope. Analysis of the profiles in the area indicates that this slope should be on the order of 1V on 10H to 1V on 15H.

Opposite the gaps, the recession of the mhwl and the slope changes were used to determine the wave heights during the 50-yr storm event. Table A6 indicates the depth-limited wave heights during this event relative to the bayward distance from the toe of the bank. These wave heights assume a worst case situation where the entire profile opposite the gap evolves to the milder slope and the horizontal berm (at +6.0 ft mlw) is substantially decreased in width.

Table A6
Depth-Limited Wave Heights Opposite Gaps

Bayward Distance From Bank Toe (ft)	Wave Height (ft)
0	< 1.0
10	2.0
20	2.5
30	3.0

Since the protection of the bank toe depends on the performance of the beach berm during design storm events, a profile response model was used to evaluate this performance. This model, developed by Kriebel and Dean (1985), calculates beach profile evolution due to storm events, and includes the effects of both water level rise and waves. The initial profile used in the simulation is the proposed beach fill configuration with the assumed equilibrium beach slope. A worst case scenario was evaluated with the model for the beach and shoreline area opposite the gaps by using the storm wave conditions prior to reduction by the offshore breakwaters. The results of this evaluation indicated that even in the worst case scenario for the 50-yr storm event, the storm berm would remain with a width of on the order of 5 to 10 ft. The actual erosion of the storm berm would be expected to be significantly less due to the reduction in the storm wave energy as a result of wave diffraction through the gaps.

Based on the preceding analyses, a gap width of 100 ft was selected for the project area.

Breakwater crest elevation

In addition to diffracted wave energy through the breakwater gaps, wave energy transmitted over the top of the structures was considered to maximize the protection of the shoreline area. This analysis was conducted using a wave transmission model developed by Ahrens (1987) capable of predicting the amount of wave energy transmitted over and through both submerged and non-submerged reef type breakwaters. Table A7 presents the results of this analysis for various combinations of breakwater crest height and slope for various return interval storms. During the 50-yr design storm, the wave heights immediately behind the breakwaters are reduced to about 60 percent, 54 percent, and 46 percent of the incident height with breakwater crest elevations of +4.0, +5.0, and +6.0 ft mlw, respectively. During the 25-year event, these reductions are 55 percent, 46 percent, and 38 percent, respectively. These transmitted waves then propagate shoreward where they are further dissipated by the beach salients formed during the evolution of the beach fill to an equilibrium planform and the storm berm. With the proposed beach fill in place, a breakwater crest elevation of + 4.0 ft mlw was selected to limit the transmitted design wave heights to about 4.0 ft (the same height as the diffracted design wave opposite the gaps) which would then be dissipated by the storm berm.

Beachfill characteristics

Seven beach profile lines were identified for sample collection. Four 1-liter samples of surface sediment were taken at locations along each profile spaced equally between the foot of the bluff and a depth of -1.0 ft, mlw. The four samples were then mixed into a composite sample for sieve analysis. These data indicate that the native beach material ranges from fine to coarse sands with a median grain size of about 0.6 mm. For optimum performance, beachfill sources with similar grain size characteristics should be used.

Summary of Breakwater and Beachfill Design Components

Based on the above analyses and evaluations, the recommended plan to accomplish the objectives of stabilizing the existing beach and providing

Table A7
Wave Transmission Versus Crest Height

Storm Event	DSWL (ft) mlw	Incident Wave Height (H (ft))	Wave Period (Tp (sec))	Transmitted Wave Heights at Various Crest Heights H(ft)		
				+4.0 mlw	+5.0 mlw	+6.0 mlw
50 yr	7	7.2	9.7	5.6	5.3	4.9
50 yr	7	6.2	9.7	4.9	4.6	4.2
50 yr	7	5	9.7	4.0	3.7	3.4
25 yr	6.2	6.5	7.6	4.8	4.4	4.0
25 yr	6.2	5.6	7.6	4.2	3.8	3.5
25 yr	6.2	4.5	7.6	3.4	3.1	2.8
10 yr	5.5	5.6	6.5	3.9	3.5	3.1
10 yr	5.5	4.8	6.5	3.4	3.0	2.7
10 yr	5.5	3.9	6.5	2.8	2.5	2.2
1 yr	4.6	5	4.3	3.1	2.7	2.3
1 yr	4.6	4.3	4.3	2.6	2.3	1.9
1 yr	4.6	3.5	4.3	2.2	1.8	1.5

wave-induced erosion control for the existing banks is the construction of 11 offshore breakwaters and the placement of beachfill. The recommended breakwater segment lengths are 100 ft each, separated by 100-ft-wide gaps (except for a 75-ft-wide gap immediately south of the existing pumping station) and located about 140 ft bayward of the existing mean high water shoreline. The recommended design crest elevation is +4.0 ft mlw.

The recommended beachfill includes a storm berm at +6.0 ft mlw extending about 30 ft bayward of the existing toe of the bank and then sloping at 1V:8H until intersecting with the existing bottom.

Project Construction

The construction of the project was initiated in November 1990 and completed in July 1991 by Coastal Design and Construction of Gloucester, Virginia. The construction sequence was breakwater construction, initial beach fill placement, extension of existing storm drains, grading and stabilization of critical bank erosion areas, and final beach fill placement.

The breakwaters were constructed by land-based equipment using temporary sand causeways from the existing shoreline out to the breakwater locations. Geotextile fabric was placed on the existing bottom followed by placement of the bedding stone directly on the filter fabric. Stone was supplied to each breakwater location via a front-end loader running between the stone stockpile areas at the north and south ends of the project area and each sand causeway. The front-end loader dumped the stone into a steel containment bin placed at the bayward end of each causeway. A backhoe was then used to remove the stone from the containment bin and place it in the breakwater section. This procedure is illustrated in Figure A4.

The first two or three breakwaters at the south end and north end of the project area were constructed initially to "anchor" the existing beach material and the intermediate beach fill. Immediately following construction of the breakwater segments, wave diffraction through the gaps began to form the salients. The pre- and post-construction shorelines are shown in Figures A5 and A6, respectively.

Construction of the project was complete in July 1991 with the final placement of beach fill. The completed project shoreline area is shown in Figures A7 and A8.

Post-Construction Monitoring

Monitoring of the project was initiated with a topographic/bathymetric survey of the project area prior to construction. Post-placement surveys of beach fill acceptance reaches were completed on July 8, 1991. Post-construction beach surveys were completed on September 28, 1991, and November 17, 1991. The purpose of these surveys was to monitor the

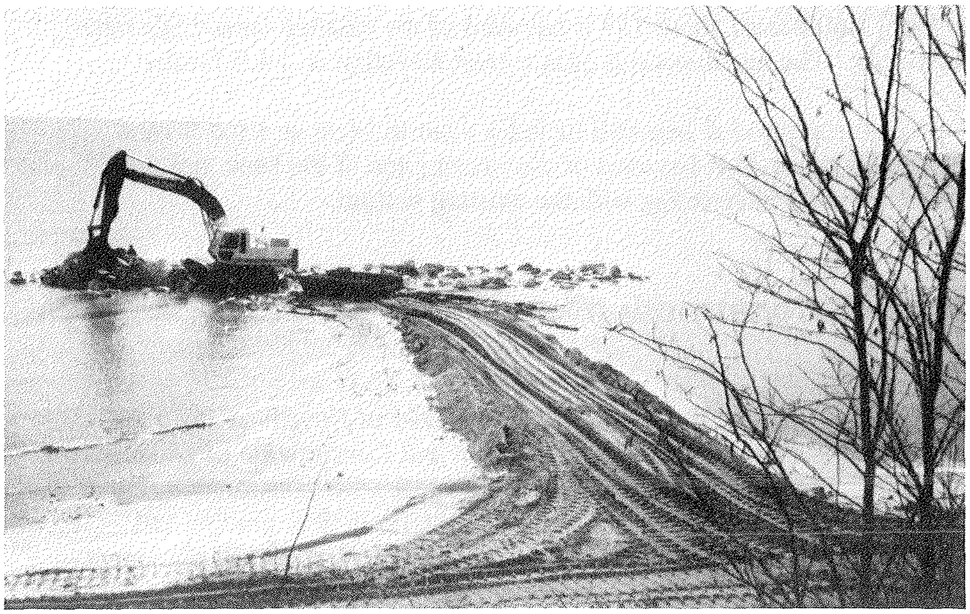


Figure A4. Breakwater construction procedure

Figure A5. Pre-construction shoreline

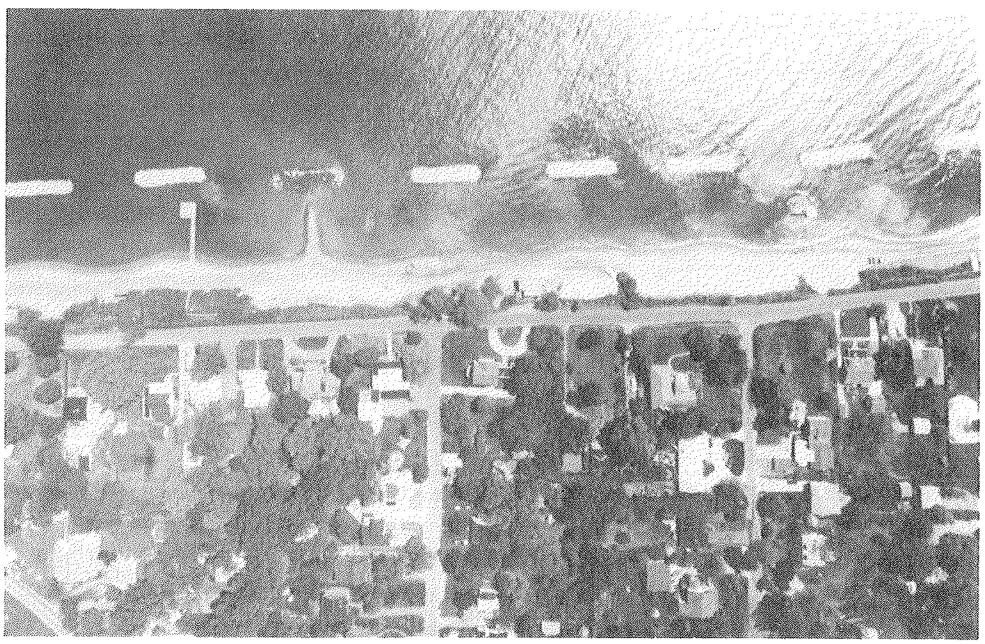


Figure A6. Post-construction shoreline

Figure A7. Completed project at south end

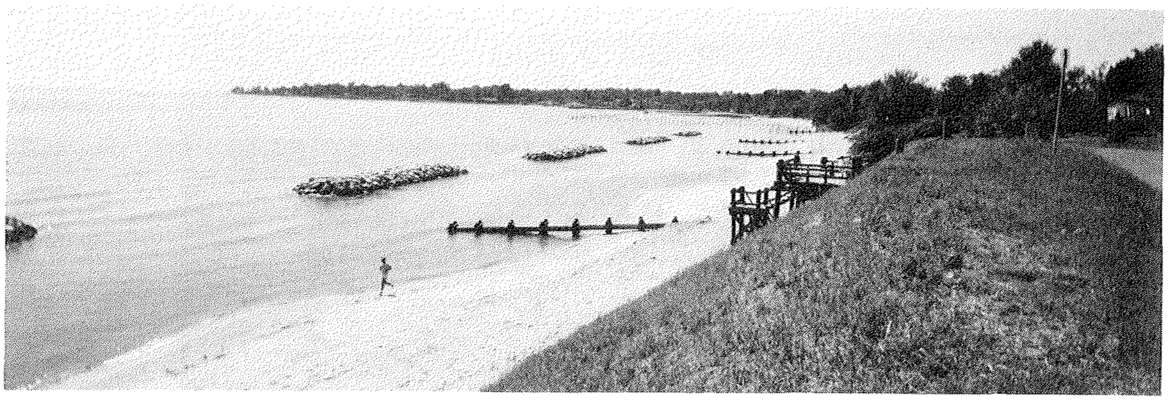


Figure A8. Completed project at north end

evolution of the beach fill as a result of the effects of the offshore breakwaters. Figure A9 shows the pre- and post-construction shoreline positions.

Wind data from the Baltimore Washington International Airport were obtained for the period January 1 to December 30, 1991, along with Littoral Environmental Observations (LEO), site photographs, and aerial photography. The wind data were used to hindcast the wave climate at the site.

The response of the shoreline following the breakwater construction and beach fill placement was initially predicted using an empirical method (Ahrens and Cox 1990). The GENESIS (GENEralized Model for SImulating Shoreline change) numerical shoreline change model (Hanson and Kraus 1989) was used to simulate the evolution of the shoreline under actual conditions that

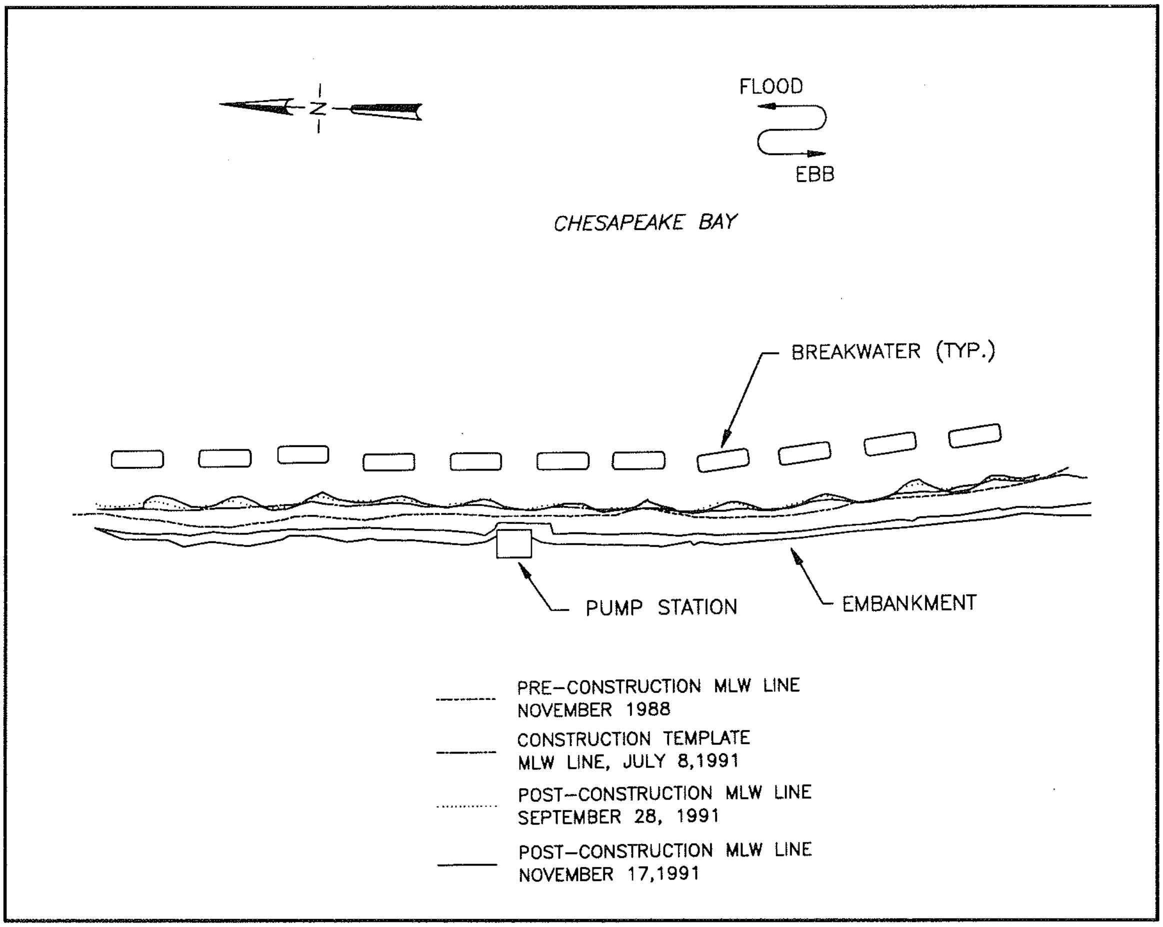


Figure A9. Pre- and post-construction shorelines

occurred since project construction. Application and results of the GENESIS modeling are presented in the following paragraphs.

GENESIS Shoreline Modeling

Model setup

The shoreline coordinate system established for the modeling is shown in Figure A10. The alongshore spacing selected was 12.5 ft to maximize the number of cells behind each detached breakwater. This spacing resulted in an average of nine cells per breakwater. Initial shoreline position data were developed based on the July 8, 1991 post-fill survey of the project area.

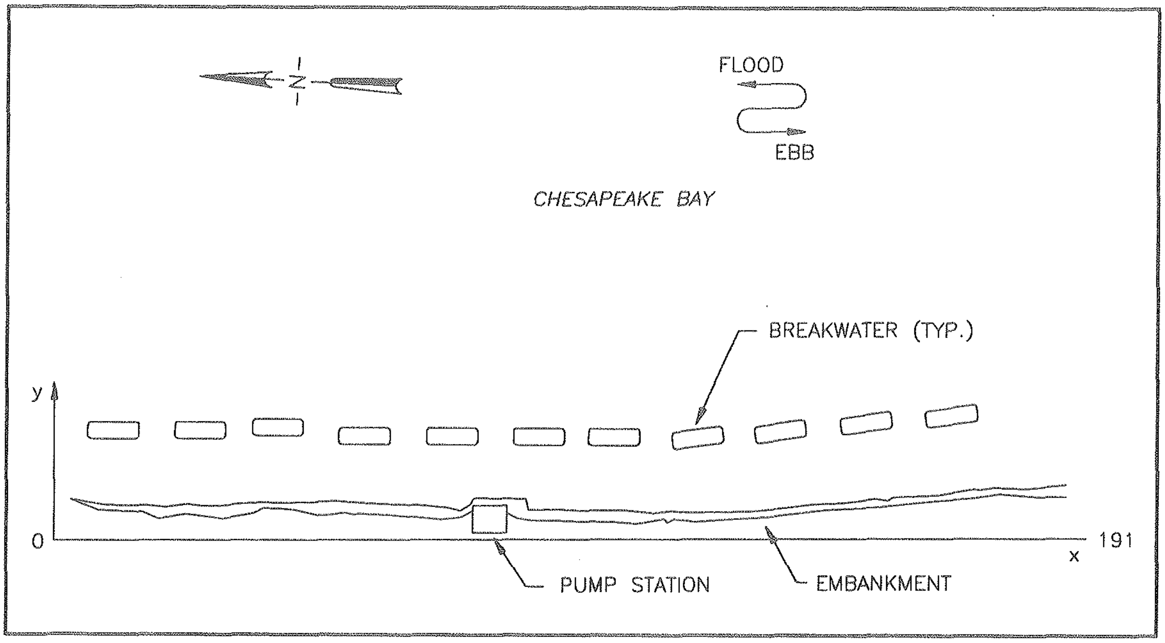


Figure A10. Shoreline coordinate system

Data for the START file

The initial model configuration is contained in START_INIT. Values for the modeling parameters were based on available data from the site and best engineering assumptions. Values selected and the rationale for their selection are discussed in the following paragraphs.

Line A.3. Breakwaters are 100 ft in length and the gaps between them are 100 ft. The exceptions are a 75-ft gap between Breakwaters No. 6 and No. 7, and Breakwater No. 11, which has a length of 75 ft.

Line A.5. Wave data were developed by hindcasting hourly wind speeds obtained from a nearby anemometer. Accordingly, the initial time interval selected is DT = 1 hr. Previous experience indicated that with an alongshore spacing of DX = 12.5 ft., this time interval should result in a reasonable stability parameter.

Line A.12. For the initial simulation, the values of K1 and K2 were left at the default values of 0.50 and 0.25, respectively. A preliminary run with these values and no offshore breakwaters resulted in a net longshore transport rate of -10,000 cu yd/yr, which compares favorably with the estimated longshore transport rate in the area.

Line B.1. For the initial simulation, the values of HCNGF, ZCNGF, and ZCNGA were set to give no change.

Line C.1. Sand placed as a part of project construction had a median grain size of 0.5 mm. This value was selected for the initial simulation.

Line C.2. The design berm elevation for the project was 6 ft above mean low water (set at the 50-yr tide elevation).

Line C.3. The depth of closure for the project area is estimated to be 8 ft based on profile analysis in the area.

Line D.1. There are no non-diffracting groins included in the simulation.

Line E.1. One diffracting groin is included at grid cell 1.

Line F.2. The bottom slope near the groins is 0.1.

Line F.3. The north groin was constructed to have low permeability.

Lines F.4 and F.5. The value of the length of the diffracting groin at grid cell 1 was taken from a survey of the area.

Lines G.6 and G.7. Locations of the breakwaters are taken from the as-built drawings of the project.

Line G.9. Transmission coefficients for the breakwaters were initially selected to be 0.10 to indicate low wave transmission.

Data for the SHORL files

The shoreline position for the initial simulation was obtained from shoreline surveys conducted on July 8, 1991.

Data for the DEPTH file

A depth file was not required because an external wave transmission model was not used.

Data for WAVES file

Wave measurements for the site for the time interval between measured shoreline positions were not available. Instead, a 1-year wave hindcast was conducted for the period January 1, 1991, through December 31, 1991. This hindcast was conducted using hourly wind data from the Baltimore/Washington International Airport, which is located about 19.5 miles northwest of Bay Ridge. Waves were hindcast up to the breakwater locations using the shallow-water wave equations in the Corps Automated Coastal Engineering System (ACES) Program, Version 1.05.

The result of this hindcast was a time series of offshore wave period, height, and direction data for the period January 1 to December 31, 1991. As a check on the acceptability of the wave data set, longshore sediment transport rates using the data were computed. This computation resulted in a predicted net longshore transport rate of -10,000 cu yd/yr, which compares favorably with the -5,000 to -10,000 cu yd/yr net transport rate calculated during the design studies and also inferred from an analysis of shoreline changes in aerial photography of the site. This good comparison supports the use of this wave data set for the modeling effort.

Calibration and verification

For the calibration and verification process for this project, the intent was to vary the values of various calibration parameters to obtain agreement between the measured shoreline of September 28, 1991 (initial beach monitoring survey) and the calculated shoreline. Once reasonable agreement was achieved between these two shorelines, the model would be verified by comparing the measured and calculated shoreline of November 17, 1991.

In the course of calibration, generally only one parameter at a time was changed in order to evaluate its effect on the calculated shoreline portion. As a first step, the value of the main parameter K_1 was varied to determine the value that would result in a calculated overall net longshore transport rate close to the previously determined values. Second, the parameter K_2 was varied to improve the agreement between the measured and calculated shoreline positions as well as the approximate magnitude of net inflow of sand from the south. Next, the longshore locations of the breakwaters were translated several grid cells to the north and south as required to improve the agreement between the calculated and measured shoreline positions. Next, the transmission coefficients of the breakwaters were varied to adjust the size of the salients behind the breakwaters. Lastly, beach fill was added to simulate the evolution of the storm berm that resulted in an increase in beach width.

In total, 15 calibration simulations were conducted. Several of the initial runs were conducted without any structures in place along the shoreline to determine the value of K_1. Evaluation of these runs indicated that $K_1 = 0.50$ resulted in a calculated net longshore transport rate of -10,000 cu yd/yr (south to north), which agreed with the previously determined rate of -5,000 to -10,000 cu yd/yr.

With $K_1 = 0.5$ and $K2 = 0.25$, an initial simulation with all breakwaters in place was conducted. Results of this run, shown in Figure A11, indicate that the bayward limit and shape of most of the salients behind the breakwaters are in reasonably good agreement with the measured salients. However, the longshore locations of the calculated salients are displaced too far to the north and the depths of the embayments are too great. The calculated Calibration Verification Error (CVE) equals 10.44 ft.

A number of additional simulations were made with the longshore locations of the breakwaters translated both north and south several cells in an attempt to improve the agreement between the longshore location of the calculated and

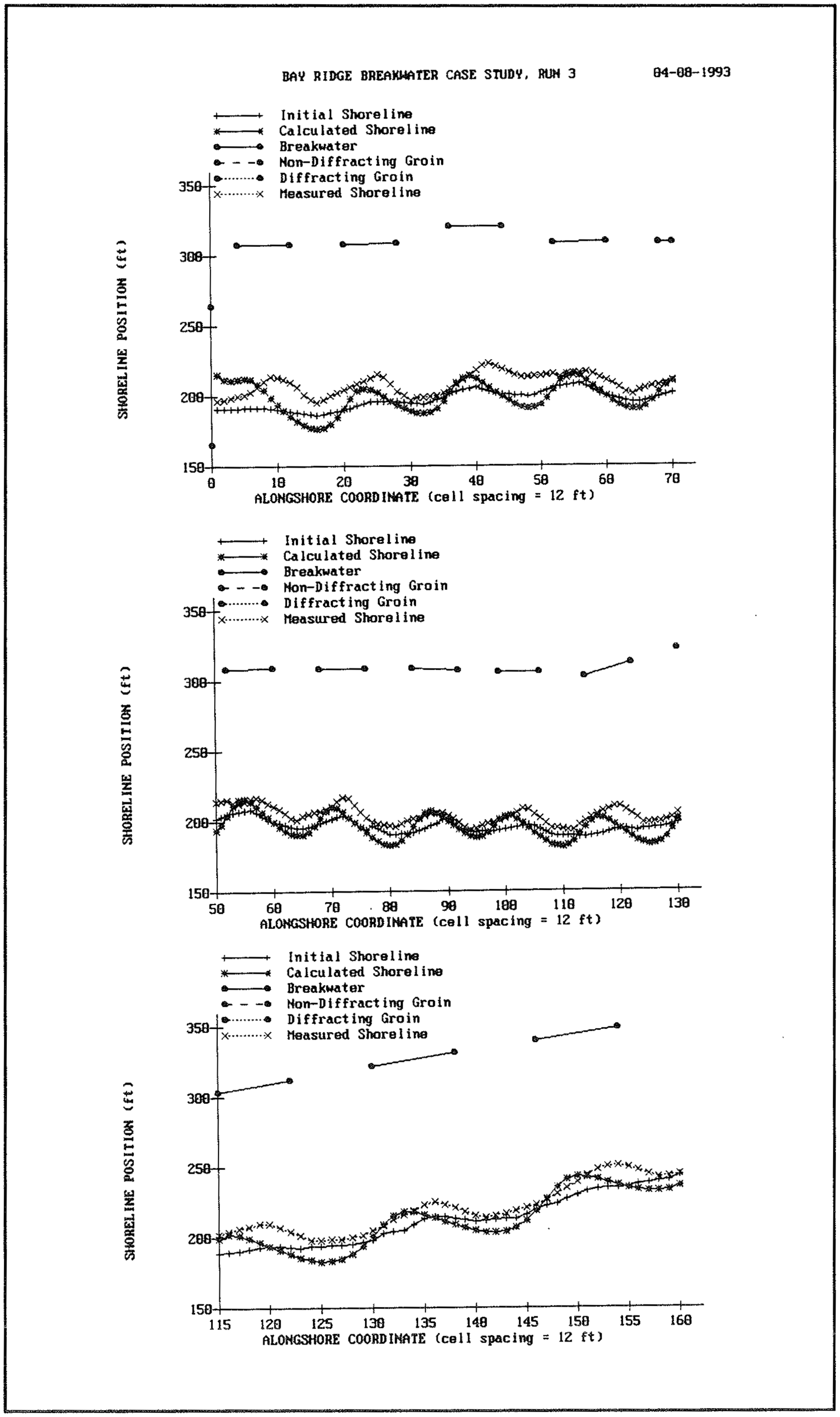


Figure A11. Initial calibration simulation measured salients. The best agreement obtained is shown in Figure A12, which has a calculated CVE equal to 9.01. As shown in Figure A12, there is an appreciable improvement in the agreement between the longshore locations of the calculated and measured salients. However, the bayward limit of the salients of the calculated shoreline needs to be increased, while the landward limit of the embayments of the calculated shoreline needs to be decreased to improve agreement with the measured shoreline.

In an attempt to increase the bayward limit of the salients of the calculated shoreline, the transmission coefficients of the breakwaters were decreased from 0.1 to 0.0, which represents no wave transmission through the breakwaters. This change had a negligible effect on the location of the salients. Next, the value of K_2 was increased from 0.25 to 0.50 and then to 0.75. The effect of these changes was an increase in the calculated CVE from 9.01 with $K_2 = 0.25$ to calculated CVE's of 9.20 and 9.88 with K2 = 0.50 and 0.75, respectively. This change also had a negligible effect on the location of the salients.

Following unsuccessful attempts at improving the agreement of the bayward limit of the salients and the landward limit of the embayments, the changes between the measured post-fill (July 8 1991) and the measured September 28, 1991 shoreline positions were analyzed in more detail. As shown in Figure A13, following the completion of the beach fill on July 8, 1991, the shoreline evolved to the position shown on September 28, 1991 as a result of the influence of the breakwaters on the wave climate. As noted in Figure A13, an overall bayward movement of the shoreline occurred, including the shoreline opposite the breakwater gaps. Although the bayward movement of the shoreline leeward of the breakwaters was expected, the bayward movement of the shoreline opposite the gaps was not anticipated. Typically, the shoreline opposite breakwater gaps evolves landward to form embayments in equilibrium with the diffracted wave climate with the sediment eroded from the embayments forming the salients or tombolos behind the breakwaters.

In this case, the bayward movement of the shoreline opposite the gaps is attributed to erosion of the storm berm constructed as a part of the beach fill. The beach fill template consisted of a 20-ft-wide berm at +6.0 ft mlw with a 1V:8H slope from the bayward edge of the berm to the existing bottom. Site visits following the beach fill placement and after some moderate storm events revealed that 1- to 3-ft-high erosion scarps had occurred along the berm opposite the breakwater gaps. The net effect was that the scarping and erosion of the berm in these areas resulted in a movement of beach fill from the berm to the offshore area to reduce the slope of the beach. As a result, the mean low water (mlw) shoreline opposite the gaps advanced bayward in all locations.

In retrospect, a straightforward application of GENESIS would not be expected to result in good agreement between the measured and calculated shorelines because of the addition of sand to the mlw beach as a result of the scarping. In an attempt to simulate this process, a simulation was made with

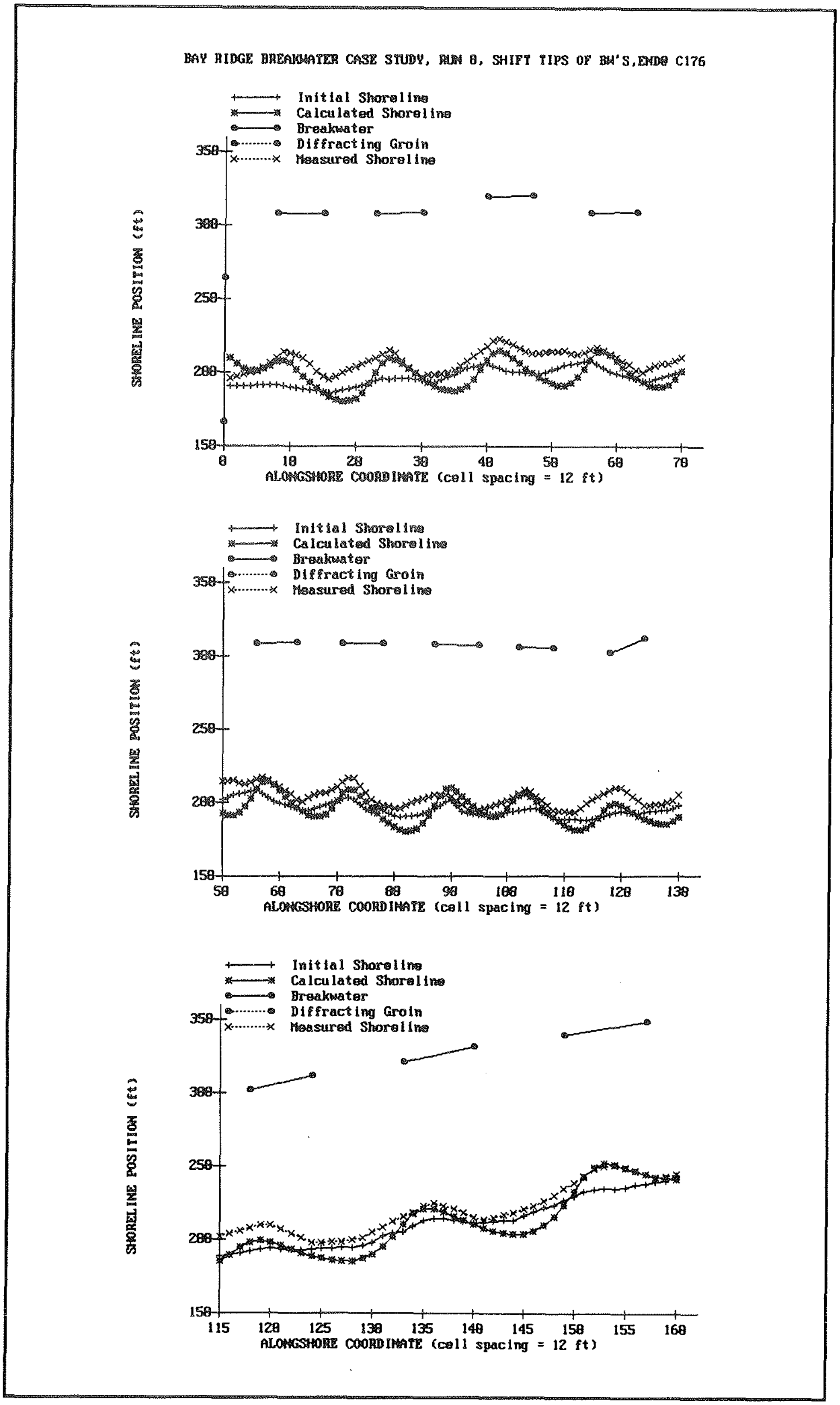


Figure A12. Calibration simulation No. 8

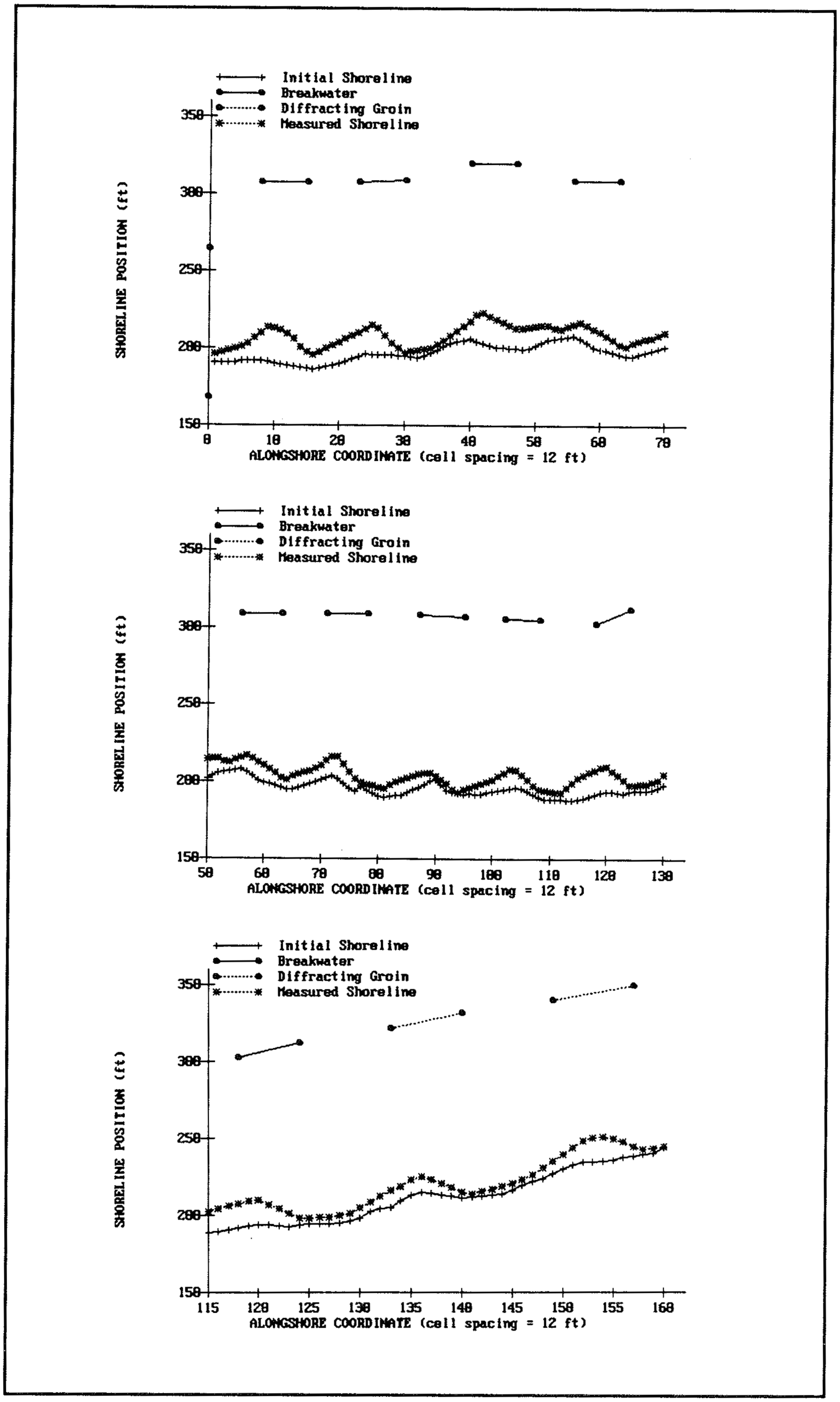


Figure A13. Measured pre- and post-fill shorelines a beach fill added between the measured post-fill shoreline on July 8, 1991 and the measured shoreline on September 28, 1991. The added berm width, YADD, was selected to be 10 ft, which was the average bayward displacement of the shoreline opposite the breakwater gaps between the two measured shorelines. The volume of the "artificial" beach fill approximated the volume of eroded material in the berm scarp.

Results of this simulation are shown in Figure A14. In general, the agreement between the measured and calculated shoreline is greatly improved with a CVE equal to 7.89.

At this point, the model was considered to be calibrated sufficiently and the verification process was initiated. The intent of this process was to use the model to reproduce a measured shoreline over a time interval independent of the calibration interval. The shoreline selected for verification of the model was the measured shoreline of November 17, 1991, since hindcast wave data were also available through that period. The model parameters used for the verification simulation were the same as for the last calibration simulation. Results of this simulation, shown in Figure A15, indicate good agreement between the measured and calculated shoreline positions, with a CVE equal to 7.51.

Summary and Discussion

The preceding sections discuss the data preparation, calibration, and verification of the GENESIS model for the Bay Ridge offshore breakwater project. A detailed description of many of the intermediate simulations is omitted.

Overall, the agreement between the measured and calculated shorelines during the calibration and verification stages is considered to be good considering the limitations of some of the data used. In particular, the wave data set was developed using wind data from an inland anemometer nearly 20 miles away from the site and hindcast techniques using the shallow-water wave equations. The use of actual wave data from the site or a more sophisticated wave hindcast would have more than likely resulted in better agreement between the measured and calculated shoreline positions. In addition, the scarping and erosion of the storm berm after initial placement, which resulted in a bayward advancement of the shoreline opposite the breakwater gaps, further complicated the modeling effort.

In any event, the agreement obtained between the measured and calculated shoreline positions even with the data limitations, clearly illustrates the capability and effectiveness of the GENESIS modeling system in simulating the influence of waves and coastal structures on the evolution of a sandy beach. The results demonstrate that the modeling system is an extremely useful engineering tool for evaluating shore protection projects.

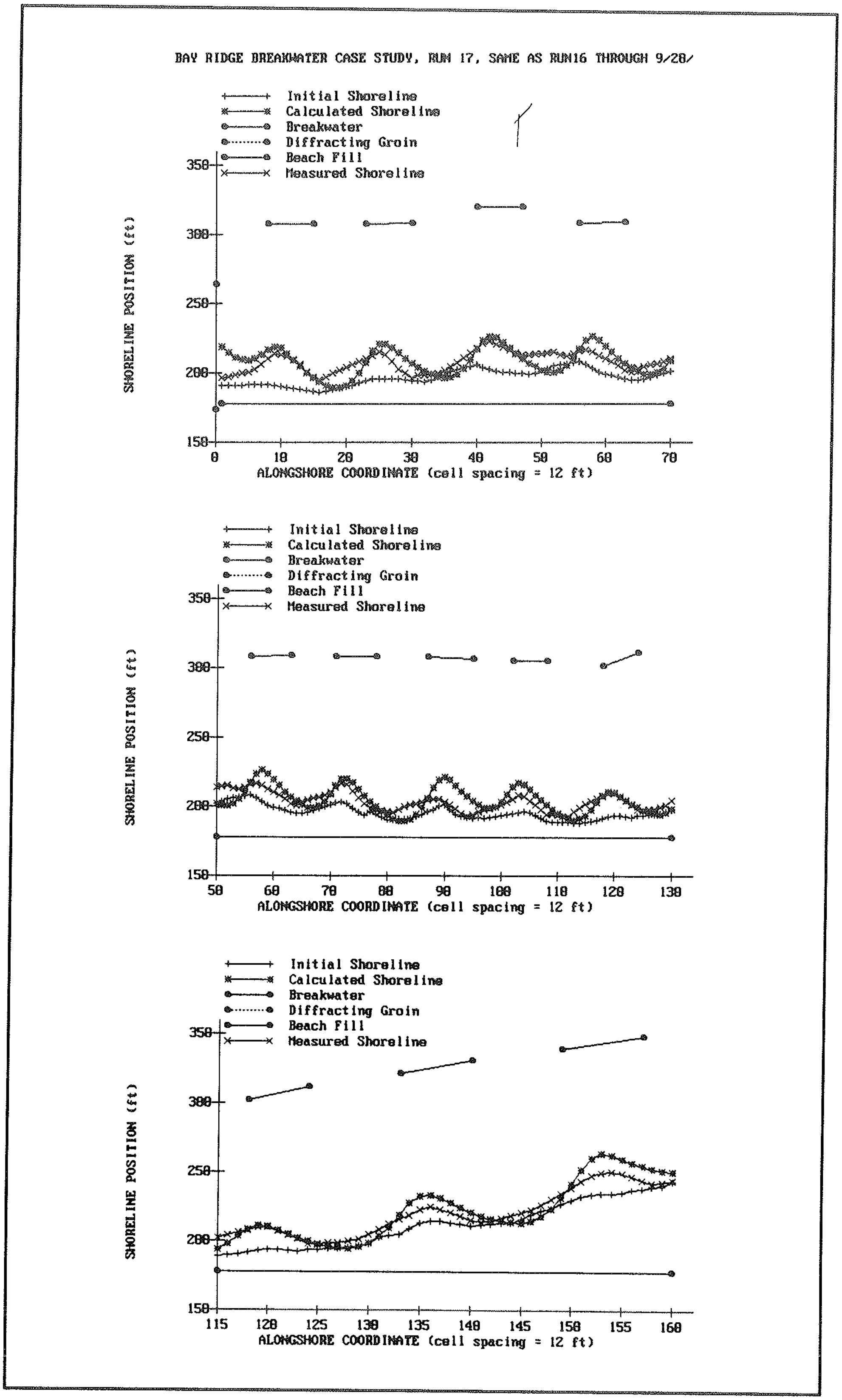


Figure A14. Final calibration simulation

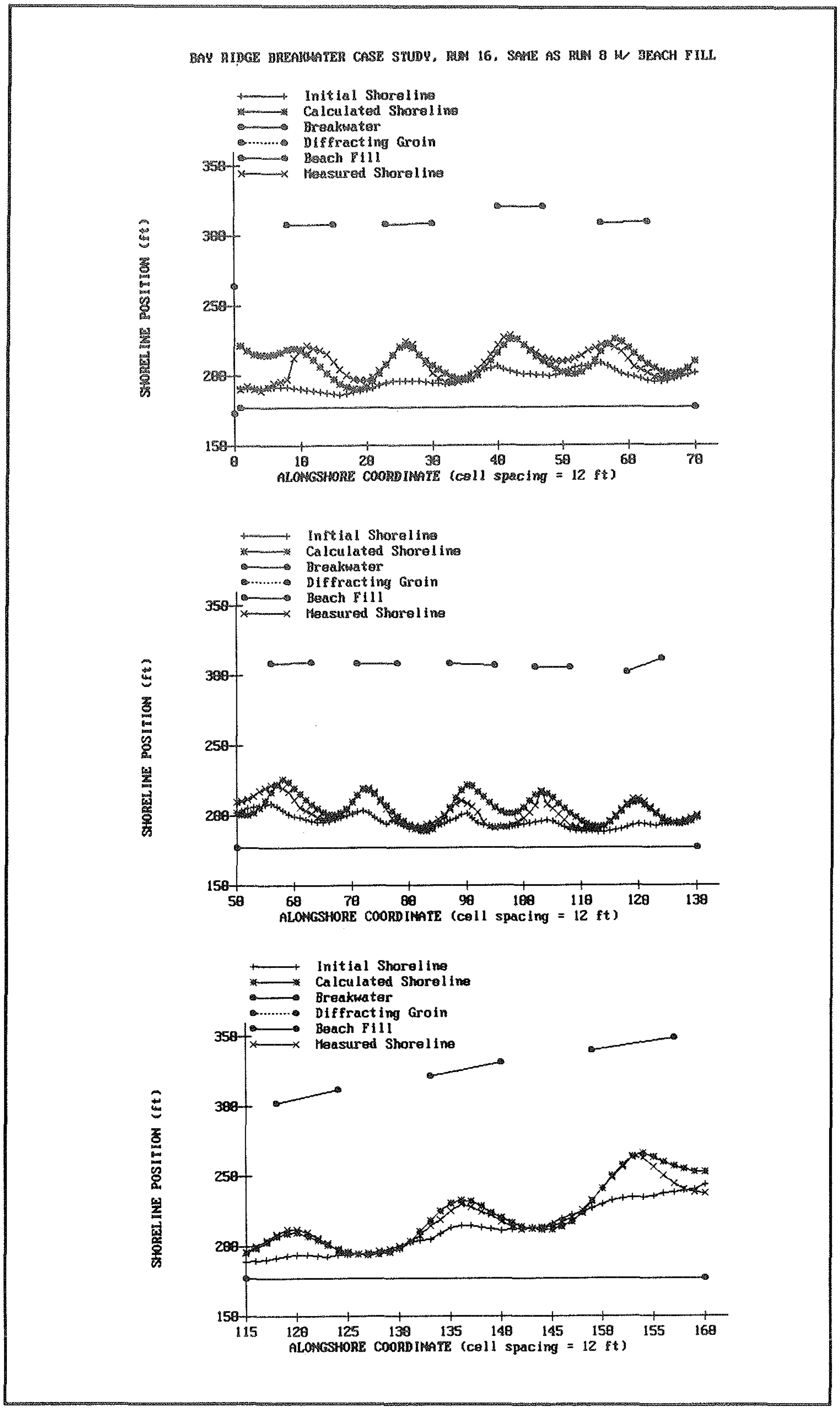


Figure A15. Verification simulation

Conclusions

To date, the Bay Ridge offshore breakwater project has performed as expected with the formation of subdued salients behind each breakwater and the resulting overall stability of the shoreline. The project has been subjected to numerous significant storm events and has prevented erosion of the bank area and roadway along the project shoreline. No adverse effects have been observed along adjacent shoreline areas. The project has been well-received by the residents of the community as a result of the stability of the shoreline and the enhancement of the recreational beach area.

References

Ahrens, J.P. (1987). "Characteristics of reef breakwaters," Technical Report CERC-87-17, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Ahrens, J. P., and Cox, J. (1990). "Design and performance of reef breakwaters," *Journal of Coastal Research,* SI #7, pp 61-75.

Hanson, H., and Kraus, N.C. (1989). "GENESIS: Generalized model for simulating shoreline change; Report 1, technical reference," Technical Report CERC-89-19, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Kriebel, D.L., and Dean, R.G. (1985). "Beach and dune response to severe storms," *Proceedings, 19th International Conference on Coastal Engineering,* American Society of Civil Engineers, pp 1584-1599.

Pope, J., and Dean, J.L. (1986). "Development of design criteria for segmented breakwaters," *Proceedings, 20th International Conference on Coastal Engineering,* November 9-14, Taipei, Taiwan. American Society of Civil Engineers, pp 2144-2158.

Reid, R.O., and Bodine, B.R. (1968). "Numerical model for storm surges, Galveston Bay," *Journal of the Waterways and Harbors Division*, American Society of Civil Engineers, 94(WW1,5805) 33-57.

Shore Protection Manual. (1984). 4th ed., 2 Vols., U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, U.S. Government Printing Office, Washington, D.C.

Appendix B Notation

a	=	Maximum indentation (headland design)
A	=	Empirical scale parameter that relates to the median beach grain size
A_e	=	Erosion area of cross-sectional profile
A_t	=	Area of breakwater cross section
b	=	Headland spacing
B_n	=	Bulk number, A_t/D_{n50}^2
C'	=	Effective slope "as built", $A_t/h_c'^2$
C_{gb}	=	Wave group speed at breaking
d	=	Depth at structure
d_g	=	Depth at gap between adjacent breakwater segment
d_s	=	Average water depth at the structure
d_{sa}	=	Depth at annual seaward limit of littoral zone
D	=	Water depth (equilibrium profile)
D_{50}	=	Mean grain size of material in project area
D_{n50}	=	Nominal diameter, $(W_{50}/\rho_a)^{1/3}$
g	=	Acceleration of gravity (9.81m/sec^2)
h	=	Water depth at toe of structure

h_c, h_c' = Armor crest level relative to seabed, after and before exposure to waves

H = Design wave height

H_b = Breaking wave height

H_i = Incident wave height

H_s = Significant wave height, average of highest one-third of the waves

H_1 = Average of highest 1 percent of all waves, $\approx 1.67\, H_s$

H_5 = Average of highest 5 percent of all waves, $\approx 1.37\, H_s$

H_{10} = Average of highest 10 percent of all waves, $\approx 1.27\, H_s$

H_{mo} = Significant wave height based on spectrum $4\sqrt{m_O}$

H_t = Transmitted wave height

H_e = Deepwater wave height exceeded 12 hr/yr

H_g = Wave height at breakwater gap

I_s = Beach response index

K_1, K_2 = Empirical coefficients

K_D = Stability coefficient

K_r = Reflection coefficient of breakwater

K_t = H_t/H_i, wave transmission coefficient

K_{to} = Overtopping transmission coefficient

K_{tt} = Through transmission coefficient

K_{π} = Through transmission coefficient

K_T = Structure transmission value

L = Wavelength at structure

L_g = Gap distance between adjacent breakwater segments

L_p = Local wave length calculated with T_p

L_s = Breakwater segment length

N = Number of waves (storm duration)

N_s = Stability number, $H_s/\Delta D_{n50}$

N_s^* = Spectral stability number, $H_{mo}/\Delta D_{n50} * s_p^{-1/3}$

p = Sand porosity

P = Structure permeability coefficient

P_{ls} = Longshore energy flux factor

Q = Longshore transport rate

Q_N = Net longshore transport rate

Q_G = Gross longshore transport rate

Q_R = Longshore transport moving to the right from an observer looking seaward

Q_L = Longshore transport moving to the left from an observer looking seaward

R = Correlation coefficient

R_1, R_2 = Radii of the spiral curve (headland design)

R_c = Crest freeboard, level of crest relative to still water

R_p^* = Dimensionless freeboard, $R_c/H_s * (s_{op}/2\pi)^{0.5}$

S = Ratio of sediment of fluid density (2.65)

S = Damage level, A_e/D_{n50}^2

S_r = Specific gravity of armor unit (ρ_a/ρ_w)

s_{op} = Fictitious wave steepness, $2\pi H_s/gT_p^2$

T_e = Wave period corresponding to H_e

T_p = Peak wave period

T_z = Average wave period

w_r = Unit weight of armor

W_{50} = Weight of the 50 percent size in the gradation

W_a = Weight of the individual armor unit

x = Longshore coordinate (Chapter 3)

x = Percentile of armor stone less than the given weight (Chapter 4)

X = Breakwater segment distance from original shoreline

X_g = Erosion/accretion opposite gap, measured from original shoreline

X_s = Salient/tombolo length in on-offshore direction measured from original shoreline

$\overline{X}$ = Effective distance offshore

y = Distance to structure from average shoreline

α = Constant angle between either radius R_1 or R_2 and its tangent to the curve

β = Predominant angle of wave approach

$tan\beta$ = Average bottom slope from the shoreline to the depth of active longshore sand transport

Δ = Relative density, $\rho_a/\rho_w - 1$

ρ_a = Mass density of armor

ρ_w = Mass density of water

ξ_z = Surf similarity parameter

θ = Angle between radii R_2 and R_1 (headland design) (Chapter 2)

θ = Angle of structure slope measured from horizontal (Chapter 4)

θ_{bs} = Angle of breaking waves to local shoreline

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. AGENCY USE ONLY *(Leave blank)*

2. REPORT DATE
December 1993

3. REPORT TYPE AND DATES COVERED
Final report

4. TITLE AND SUBTITLE
Engineering Design Guidance for Detached Breakwaters as Shoreline Stabilization Structures

5. FUNDING NUMBERS
WU 32748

6. AUTHOR(S)
Monica Chasten, Julie D. Rosati, John W. McCormick,
Robert E. Randall

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
USAE Waterways Experiment Station, Coastal Engineering Research Center, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199
Texas A&M University, Ocean Engineering Program, Civil Engineering Department, College Station, TX 77843

8. PERFORMING ORGANIZATION REPORT NUMBER
Technical Report
CERC-93-19

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
U.S. Army Corps of Engineers
Washington, DC 20314-1000

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161

12a. DISTRIBUTION/AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

12b. DISTRIBUTION CODE

13. ABSTRACT *(Maximum 200 words)*

Detached breakwaters can be a viable method of shoreline stabilization and protection in the United States. Breakwaters can be designed to retard erosion of an existing beach, promote natural sedimentation to form a new beach, increase the longevity of a beach fill, and maintain a wide beach for storm damage reduction and recreation. The combination of low-crested breakwaters and planted marsh grasses is increasingly being used to establish wetlands and control erosion along estuarine shorelines.

This report summarizes and presents the most recent functional and structural design guidance available for detached breakwaters and provides examples of both prototype projects and the use of available tools to assist in breakwater design. Functional design guidance presented includes a review of existing analytical techniques and design procedures, functional design considerations, and data requirements. The chapter on structural design guidance includes static and dynamic breakwater stability and methods to determine performance characteristics such as transmission, reflection, and energy dissipation. Also included is a discussion of numerical and physical modeling as tools for prediction of morphological response to detached breakwaters, and a case example of a breakwater project designed and constructed at Bay Ridge, Maryland.

14. SUBJECT TERMS

Beach Stabilization	Salient
Breakwaters	Tombolo

15. NUMBER OF PAGES
167

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
UNCLASSIFIED	UNCLASSIFIED		